The feasibility of a commercial osmotic power plant

Appendices

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Nomenclature

Symbols

%	percentage	[-]
A	area or cross section	m^2
A_m	membrane area	m^2
A_w	water permeation coefficient	m/s·Pa
c	molar concentration	mol/m^3
C	costs	€
D	diameter	m
d	thickness	m
E	energy	J
F	Faraday constant	96.485 C/mol
f	friction coefficient	[-]
g	acceleration of gravity	9.81 m/s^2
G	Gibbs free energy	J
h	head	M
H	Height	m
J	flux	mol/m^2s
k	resistance to salt diffusion through porous substrate	s/m
k_N	roughness	m
L	length	m
M	molar mass	kg/mol
m	mass	kg
n	number	[-]
p	pressure	Pa
P	power	W
$p.d$	packing density	m^2/m^3
Q	discharge or flow rate	m^3/s
R	universal gas constant	8.314 J/mol·K
Re	Reynolds number	[-]
S	salinity	g/l
t	time	s
T	absolute temperature	K
u	velocity	m/s
V	volume	m^3
W	power density	W/m^2
w	width	m
x	mole fraction	[-]
z	static head	m
z	valence of ions (in equation A.3)	[-]

Greek letters

γ	specific weight	kN/m^3
γ_g	safety factor	[-]
Δ	difference	[-]
η	efficiency	[-]
μ	molar free energy	J/mol
ν	kinematic viscosity	m^2/s
ξ	local loss factor	[-]
π	osmotic pressure	Pa
ρ	density	kg/m^3
$\bar{\upsilon}$	partial molar volume at given temperature and pressure	m^3/mol
φ	electrochemical potential	V

Subscripts

$€/m^2$	unit rate per area
$€/m^3$	unit rate per volume
$^\circ C$	Celcius
b	brackish solution
$buoy$	buoyancy
c	concentrated solution
Cl	chloride
d	diluted solution
eff	effective
eq	equivalent
f	friction
H_2O	water molecule
hrs/yr	hours per year
i	component
K	Kelvin
L	local
$loss$	losses
max	maximum
mix	mixture
Na	sodium
$NaCl$	sodium chloride molecule
opt	optimal
osm	osmotic
$R.O.T.$	rule of thumb
sec/yr	seconds per year
sup	supports
tur	turbine
w	water

Superscripts

0	standard conditions
PRO	pressure retarded osmosis

Appendix A
Osmotic calculations

A.1 Parameters

The practical osmotic energy depends on a number of parameters. Some of these parameters vary over the year, so these parameters depend on data. In the case study, the data is used of the following monitoring locations [1]:

Figure A.1: Monitoring locations.

A.1.1 Salinity

The salinity of the fresh and salt water varies over the year. The data of the salt water salinity is given in table A.1:

	Salinity salt [g/l]
Jan	30.40
Feb	30.50
Mar	27.60
Apr	29.50
May	29.80
Jun	30.00
Jul	30.40
Aug	30.70
Sep	30.00
Oct	30.60
Nov	30.10
Dec	29.80
Avg	29.95

	Salinity salt [g/l]
Jan	26.71
Feb	30.40
Mar	29.36
Apr	27.01
May	28.00
Jun	27.09
Jul	27.59
Aug	28.21
Sep	27.90
Oct	27.47
Nov	28.92
Dec	28.92
Avg	28.13

Table A.1: The average salt water salinity at Scharendijke diepe put (left) and Ter Heide 2 km offshore (right).

The data of the fresh water salinity at the Haringvliet outlet sluices is given in table A.2:

	Salinity fresh [g/l]
Jan	0.38
Feb	0.28
Mar	0.25
Apr	0.25
May	0.30
Jun	0.28
Jul	0.28
Aug	0.32
Sep	0.28
Oct	0.30
Nov	0.38
Dec	0.35
Avg	0.30

Table A.2: The average fresh water salinity at the Haringvliet outlet sluices.

A.1.2 Temperature

The temperature of the fresh and salt water varies over the year. The data of the temperature is given in table A.3

	Temp. fresh [°C]
Jan	4.53
Feb	5.90
Mar	7.30
Apr	11.78
May	15.27
Jun	18.90
Jul	20.03
Aug	21.17
Sep	18.42
Oct	13.70
Nov	9.93
Dec	6.63
Avg	12.80

	Temp. salt [°C]
Jan	2.87
Feb	2.24
Mar	4.59
Apr	8.48
May	12.91
Jun	16.93
Jul	19.02
Aug	19.08
Sep	16.43
Oct	13.03
Nov	8.41
Dec	4.88
Avg	10.74

	Temp. salt [°C]
Jan	5.7
Feb	5.4
Mar	5.8
Apr	8.2
May	11.6
Jun	15.4
Jul	17.9
Aug	19.4
Sep	17.9
Oct	14.6
Nov	11.1
Dec	7.8
Avg	11.7

Table A.3: Water temperature at Haringvliet outlet sluices (left), Scharendijke diepe put (middle) and Noordwijk meetpost (right).

The temperature in Kelvin can be calculated by using equation A.1:

$$T_K = T_{^\circ C} + 273.15 \tag{A.1}$$

A.1.3 Molar mass

The molar mass [g/mol] of a NaCl molecule is constant and is equal to:

$$M_{NaCl} = M_{Na^+} + M_{Cl^-} = 22.99 + 35.45 = 58.44 \tag{A.2}$$

A.1.4 Universal gas constant

The universal gas constant R is constant and is equal to 8.314 J/mol·K.

A.2 Pressure retarded osmosis (PRO)

A.2.1 Derivation of the Van't Hoff equation

The molar free energy of a solution is given by:

$$\mu_i = \mu_i^0 + \bar{\upsilon}_i \cdot \Delta p + R \cdot T \cdot \ln\left(x_i\right) + \left|z_i\right| \cdot F \cdot \Delta\varphi \qquad \textbf{(A.3)}$$

For PRO, the gradient in molar free energy is given by:

$$\begin{aligned} \Delta\mu_{H_2O} &= \mu_{H_2O,c} - \mu_{H_2O,d} \\ \bar{\upsilon}_{H_2O,c} \cdot \Delta p_c + R \cdot T \cdot \ln\left(x_{H_2O,c}\right) &= R \cdot T \cdot \ln\left(x_{H_2O,d}\right) \end{aligned} \qquad \textbf{(A.4)}$$

This gradient ensures an osmotic flow. During the osmotic flow, water molecules are transported through the semi-permeable membranes. This transport will change the molar fraction on either side of the membrane, and a hydraulic pressure difference across the semi-permeable membrane is built up. Equation A.4 can be redefined as:

$$\begin{aligned} \Delta p_c &= \frac{R \cdot T}{\bar{\upsilon}_{H_2O,c}} \cdot \left(\ln\left(x_{H_2O,d}\right) - \ln\left(x_{H_2O,c}\right)\right) \\ \Delta p_c &= \frac{R \cdot T}{\bar{\upsilon}_{H_2O,c}} \cdot \ln\left(\frac{x_{H_2O,d}}{x_{H_2O,c}}\right) \end{aligned} \qquad \textbf{(A.5)}$$

For practical reasons, equation A.5 is expressed in terms of a solute molar fraction. This implies that all the terms related to H_2O must change into terms related to a NaCl molecule:

$$\begin{aligned} \ln\left(x_{H_2O}\right) &= \ln\left(1 - 2 \cdot x_{NaCl}\right) \approx 2 \cdot \ln\left(1 - x_{NaCl}\right) \\ \Delta p_c &= \frac{2 \cdot R \cdot T}{\bar{\upsilon}_{NaCl}} \cdot \ln\left(\frac{1 - x_{NaCl,d}}{1 - x_{NaCl,c}}\right) \end{aligned} \qquad \textbf{(A.6)}$$

When the osmotic pressure is to be calculated for a sodium chloride solution, the concentration of NaCl has to be multiplied with 2 because of the dissociation into two ions (a sodium ion and a chloride ion). Both ions affect the osmotic pressure. In the general case, the ion concentration per dissociated molecule is given as the symbol i.

For small molar fractions of the solutes, the following applies:

$$\ln\left(1-x_{NaCl,d}\right)\approx -x_{NaCl,d} \qquad \textbf{(A.7)}$$

Equation A.6 can now be simplified by expressing the solute molar fractions in solute concentrations:

$$\Delta p_c = \frac{2\cdot R\cdot T}{\bar{\upsilon}_{NaCl}}\cdot \ln\left(1-x_{NaCl,d}\right)-\ln\left(1-x_{NaCl,c}\right)$$
$$\Delta p_c = \frac{2\cdot R\cdot T}{\bar{\upsilon}_{NaCl}}\cdot\left(x_{NaCl,c}-x_{NaCl,d}\right) \qquad \textbf{(A.8)}$$

By multiplying with the specific density, the solute concentrations are obtained:

$$\Delta p_c = \frac{2\cdot R\cdot T}{\bar{\upsilon}_{NaCl}}\cdot\left(x_{NaCl,c}-x_{NaCl,d}\right)$$

$$\bar{\upsilon}_{NaCl}=\frac{M_{NaCl}}{\rho} \qquad x_{NaCl}=\frac{S_{NaCl}}{\rho} \qquad \textbf{(A.9)}$$

$$\Delta p_c = \frac{2\cdot R\cdot T}{M_{NaCl}}\cdot\left(S_{NaCl,c}-S_{NaCl,d}\right)$$

The osmotic flow continues until equilibrium is reached. The pressure difference across the membrane at equilibrium is also known as the osmotic pressure difference:

$$\Delta\pi_{osm} = \frac{2\cdot R\cdot T}{M_{NaCl}}\cdot\left(S_{NaCl,c}-S_{NaCl,d}\right) \qquad \textbf{(A.10)}$$

Equation A.10 is called the van't Hoff equation.

A.2.2 Theoretical osmotic pressure difference

The theoretical osmotic pressure difference with a varying salinity (see section A.1.1) and temperature (see section A.1.2) can now be calculated by using the Van't Hoff equation:

$$\Delta\pi_{osm} = \frac{2\cdot R}{M_{NaCl}}\cdot\left(T_c\cdot S_{NaCl,c}-T_d\cdot S_{NaCl,d}\right) \qquad \textbf{(A.11)}$$

The results of the theoretical osmotic pressure difference at the Haringvliet – Grevelingen location are given in table A.4:

	Salinity salt [g/l]	Salinity fresh [g/l]	Temperature salt [°C]	Temperature fresh [°C]	Concentration salt [mol/l]	Concentration fresh [mol/l]	Δ Osmotic pressure [bar/m3]
Jan	30.40	0.38	2.87	4.53	0.5202	0.0065	23.58
Feb	30.50	0.28	2.24	5.90	0.5219	0.0048	23.68
Mar	27.60	0.25	4.59	7.30	0.4723	0.0043	21.61
Apr	29.50	0.25	8.48	11.78	0.5048	0.0043	23.44
May	29.80	0.30	12.91	15.27	0.5099	0.0051	24.01
Jun	30.00	0.28	16.93	18.90	0.5133	0.0048	24.53
Jul	30.40	0.28	19.02	20.03	0.5202	0.0048	25.04
Aug	30.70	0.32	19.08	21.17	0.5253	0.0055	25.26
Sep	30.00	0.28	16.43	18.42	0.5133	0.0048	24.49
Oct	30.60	0.30	13.03	13.70	0.5236	0.0051	24.67
Nov	30.10	0.38	8.41	9.93	0.5150	0.0064	23.81
Dec	29.80	0.35	4.88	6.63	0.5099	0.0060	23.30
Avg	29.95	0.30	10.74	12.80	0.51	0.01	23.95

Table A.4: Theoretical osmotic pressure difference at the Haringvliet – Grevelingen location.

The results of the theoretical osmotic pressure difference at STP Houtrust are given in table A.5:

	Salinity salt [g/l]	Salinity fresh [g/l]	Temperature salt [°C]	Temperature fresh [°C]	Concentration salt [mol/l]	Concentration fresh [mol/l]	Δ Osmotic pressure [bar/m3]
Jan	26.71	2	5.7	4.53	0.4570	0.0342	19.61
Feb	30.40	2	5.4	5.90	0.5202	0.0342	22.51
Mar	29.36	2	5.8	7.30	0.5025	0.0342	21.71
Apr	27.01	2	8.2	11.78	0.4622	0.0342	20.00
May	28.00	2	11.6	15.27	0.4792	0.0342	21.05
Jun	27.09	2	15.4	18.90	0.4636	0.0342	20.58
Jul	27.59	2	17.9	20.03	0.4722	0.0342	21.18
Aug	28.21	2	19.4	21.17	0.4827	0.0342	21.81
Sep	27.90	2	17.9	18.42	0.4774	0.0342	21.45
Oct	27.47	2	14.6	13.70	0.4701	0.0342	20.86
Nov	28.92	2	11.1	9.93	0.4948	0.0342	21.78
Dec	28.92	2	7.8	6.63	0.4948	0.0342	21.53
Avg	28.13	0.30	11.7	12.80	0.48	0.034	21.17

Table A.5: Theoretical osmotic pressure difference at STP Houtrust.

A.2.3 Practical osmotic pressure difference

The theoretical osmotic pressure difference cannot be used completely. The optimum power density for a semi-permeable membrane is achieved when only half the effective osmotic pressure difference is used:

$$W_{opt}^{PRO} = A_w \frac{\Delta\pi_{eff}^2}{4}$$
$$\Delta p = \frac{1}{2}\Delta\pi_{eff} \qquad \textbf{(A.12)}$$

The term Δp is the hydrostatic pressure difference over the membrane. This term indicates the pressure which is actually used for electricity generation. From now on, this term will be called practical osmotic pressure difference. The practical osmotic pressure difference is equal to half the effective osmotic pressure difference. The effective osmotic pressure difference can be calculated using:

$$\Delta\pi_{eff} = \pi_c - \pi_d \cdot \exp\left(J_w \cdot k\right) \qquad \textbf{(A.13)}$$

In the case studies, it is assumed that the effective osmotic pressure difference is equal to:

$$\begin{aligned} \Delta\pi_{eff} &= \pi_c - \pi_d \\ \exp\left(J_w \cdot k\right) &\approx 1 \end{aligned} \qquad \textbf{(A.14)}$$

The value of the exponent is assumed to be equal to 1, because the order of the water flux is 10^{-6} m/s and the order of the resistance to salt diffusion through porous substrate is 10^{-5} s/m [2].

The equation for the effective and practical osmotic pressure difference changes to:

$$\begin{aligned} \Delta\pi_{eff} &= \pi_c - \pi_d \\ \Delta p &= \frac{1}{2}\Delta\pi_{eff} = \frac{1}{2}\left(\pi_c - \pi_d\right) \end{aligned} \qquad \textbf{(A.15)}$$

The results of the practical osmotic pressure difference for the Haringvliet – Grevelingen location are given in table A.6:

	Theoretical Δ Osmotic pressure [bar/m3]	**Practical Δ Osmotic pressure [bar/m3]**
Jan	23.58	11,79
Feb	23.68	11,84
Mar	21.61	10,81
Apr	23.44	11,72
May	24.01	12,00
Jun	24.53	12,26
Jul	25.04	12,52
Aug	25.26	12,63
Sep	24.49	12,24
Oct	24.67	12,34
Nov	23.81	11,91
Dec	23.30	11,65
Avg	23.95	11,98

Table A.6: Theoretical osmotic pressure difference at STP Houtrust.

The results of the practical osmotic pressure difference for STP Houtrust are given in table A.7:

	Theoretical Δ Osmotic pressure [bar/m3]	Practical Δ Osmotic pressure [bar/m3]
Jan	19.61	9.81
Feb	22.51	11.26
Mar	21.71	10.86
Apr	20.00	10.00
May	21.05	10.53
Jun	20.58	10.29
Jul	21.18	21.18
Aug	21.81	10.91
Sep	21.45	10.73
Oct	20.86	10.43
Nov	21.78	10.89
Dec	21.53	10.77
Avg	21.17	10.59

Table A.7: Theoretical osmotic pressure difference at STP Houtrust.

A.2.4 Practical osmotic energy

The practical osmotic energy that can be recovered from 1 cubic meter of fresh water can be calculated by using equation A.16:

$$E_{osm} = \Delta p \cdot 10^{1} \ [\mathrm{MJ}] \qquad \textbf{(A.16)}$$

The results for both locations are given in table A.8:

	Haringvliet – Grevelingen [MJ/m^3]	STP Houtrust [MJ/m^3]
Jan	1.18	0.98
Feb	1.18	1.13
Mar	1.08	1.09
Apr	1.17	1.00
May	1.20	1.05
Jun	1.23	1.03
Jul	1.25	1.06
Aug	1.26	1.09
Sep	1.22	1.07
Oct	1.23	1.04
Nov	1.19	1.09
Dec	1.16	1.08
Avg	1.20	1.06

Table A.8: Theoretical osmotic pressure difference at STP Houtrust.

A.3 Reversed electro dialysis (RED)

A.3.1 Theoretical osmotic energy

The theoretical osmotic energy in case of a RED power plant is determined by using the Gibbs free energy of mixing equation:

$$\Delta_{mix}G = \Delta G_c + \Delta G_d - \Delta G_b \qquad \textbf{(A.17)}$$

Equation A.17 represents the difference in free energy between a brackish, concentrated and diluted solution:

$$\Delta_{mix}G = \sum c_{i,b} \cdot R \cdot T \cdot \ln\left(x_{i,b}\right) - \left(c_{i,c} \cdot R \cdot T \cdot \ln\left(x_{i,c}\right) - c_{i,d} \cdot R \cdot T \cdot \ln\left(x_{i,d}\right)\right) \qquad \textbf{(A.18)}$$

Equation A.18 should be calculated for both Na^+ and Cl^-. The results of the Haringvliet – Grevelingen location are given in table A.9:

	ΔG_b [MJ/m³]	ΔG_c [MJ/m³]	ΔG_d [MJ/m³]	Theoretical osmotic energy [MJ/m³]
Jan	-1.56	-0.15	-3.24	1.52
Feb	-1.55	-0.12	-3.24	1.57
Mar	-1.64	-0.11	-3.17	1.43
Apr	-1.62	-0.11	-3.28	1.55
May	-1.63	-0.13	-3.34	1.57
Jun	-1.65	-0.12	-3.39	1.61
Jul	-1.65	-0.12	-3.42	1.64
Aug	-1.64	-0.14	-3.43	1.65
Sep	-1.65	-0.12	-3.38	1.61
Oct	-1.61	-0.13	-3.35	1.61
Nov	-1.60	-0.15	-3.29	1.54
Dec	-1.59	-0.14	-3.24	1.51
Avg	-1.62	-0.13	-3.31	1.57

Table A.9: Theoretical osmotic energy at the Haringvliet – Grevelingen location.

The results of STP Houtrust are given in table A.10:

	ΔG_b [MJ/m³]	ΔG_c [MJ/m³]	ΔG_d [MJ/m³]	Theoretical osmotic energy [MJ/m³]
Jan	-1.66	-0.53	-3.19	1.00
Feb	-1.57	-0.54	-3.30	1.19
Mar	-1.60	-0.54	-3.28	1.14
Apr	-1.67	-0.55	-3.26	1.04
May	-1.67	-0.55	-3.33	1.10
Jun	-1.71	-0.56	-3.34	1.07
Jul	-1.71	-0.56	-3.38	1.10
Aug	-1.71	-0.57	-3.41	1.14
Sep	-1.71	-0.56	-3.38	1.11
Oct	-1.70	-0.55	-3.32	1.07
Nov	-1.65	-0.54	-3.32	1.13
Dec	-1.63	-0.54	-3.28	1.12
Avg	-1.67	-0.55	-3.32	1.10

Table A.10: Theoretical osmotic energy at STP Houtrust.

A.3.2 Practical osmotic energy

The optimum power density is obtained when only half of the practical osmotic energy is used. The practical osmotic energy is given in table A.11:

	Theoretical osmotic energy [MJ/m^3]	Practical osmotic energy [MJ/m^3]
Jan	1.52	0.76
Feb	1.57	0.78
Mar	1.43	0.71
Apr	1.55	0.78
May	1.57	0.79
Jun	1.61	0.81
Jul	1.64	0.82
Aug	1.65	0.83
Sep	1.61	0.80
Oct	1.61	0.81
Nov	1.54	0.77
Dec	1.51	0.76
Avg	1.57	0.78

Table A.11: Theoretical osmotic energy at the Haringvliet – Grevelingen location.

The practical osmotic energy for STP Houtrust is given in table A.12:

	Theoretical osmotic energy [MJ/m^3]	Practical osmotic energy [MJ/m^3]
Jan	1.00	0.50
Feb	1.19	0.59
Mar	1.14	0.57
Apr	1.04	0.52
May	1.10	0.55
Jun	1.07	0.54
Jul	1.10	0.55
Aug	1.14	0.57
Sep	1.11	0.56
Oct	1.07	0.54
Nov	1.13	0.56
Dec	1.12	0.56
Avg	1.10	0.55

Table A.12: Theoretical osmotic energy at STP Houtrust.

A.4 Required flow rates

A.4.1 PRO

The required fresh water flow rate through the power plant depends on the practical osmotic energy and the capacity of the power plant:

$$Q_{fresh;\,power\ plant} = \frac{P_{power\ plant}}{E_{osm}} \tag{A.19}$$

However, 90% of the inflowing fresh water will permeate through the membranes. The remaining 10%, the so-called fresh water bleed, will be transported back to the start of the production process and can be reused again. This implies that the required fresh water flow rate through the intake can be reduced by 10%. This results in a required intake flow rate equal to:

$$Q_{fresh;intake} = 0.9 \cdot \frac{P_{power\ plant}}{E_{osm}} \qquad \textbf{(A.20)}$$

The ratio between the fresh and salt water flow is 1:2, so the required salt water flow is two times the fresh water flow rate through the power plant:

$$Q_{salt} = 2 \cdot Q_{fresh;\,power\ plant} \qquad \textbf{(A.21)}$$

The required brackish water discharge rate is equal to the permeated water flow rate and the salt water flow rate through the power plant:'

$$Q_{brackish} = 0.9 \cdot Q_{fresh;\,power\ plant} + Q_{salt} = 2.9 \cdot Q_{fresh;\,power\ plant} \qquad \textbf{(A.22)}$$

A.4.2 RED

In the case of a RED power plant, no fresh water bleed will occur so the required fresh water flow rate through the power plant is equal to the required intake flow rate and can be determined with equation A.23.

$$Q_{fresh} = \frac{P_{power\ plant}}{E_{osm}} \qquad \textbf{(A.23)}$$

The ratio between the fresh and salt water flow rate is in the case of a RED power plant equal to 1:1:

$$Q_{salt} = Q_{fresh} \qquad \textbf{(A.24)}$$

The required brackish water discharge is equal to the summation of the fresh and salt water flow rate:

$$Q_{brackish} = Q_{fresh} + Q_{salt} = 2 \cdot Q_{fresh} \qquad \textbf{(A.25)}$$

A.4.3 Results

The required flow rates in case of a 25 MW PRO power plant are given in table A.13:

	Osmotic energy [MJ/m^3]	Q_{fresh} [m^3/s]	$Q_{fresh.intake}$ [m^3/s]	Q_{salt} [m^3/s]	$Q_{brackish}$ [m^3/s]
Jan	1.18	21.21	19.09	42.42	61.50
Feb	1.18	21.12	19.01	42.24	61.24
Mar	1.08	23.14	20.82	46.27	67.09
Apr	1.17	21.33	19.20	42.67	61.87
May	1.20	20.83	18.74	41.65	60.39
Jun	1.23	20.38	18.35	40.77	59.11
Jul	1.25	19.97	17.97	39.94	57.91
Aug	1.26	19.79	17.82	39.59	57.40
Sep	1.22	20.42	18.38	40.84	59.22
Oct	1.23	20.27	18.24	40.53	58.77
Nov	1.19	21.00	18.90	42.00	60.89
Dec	1.16	21.46	19.32	42.93	62.24
Avg	1.20	20.91	18.82	41.82	60.64

Table A.13: Required flow rates in case of the 25 MW PRO power plant.

Appendix B
Intake and outfall systems

B.1 Parameters

B.1.1 Specific density

The used values for the specific density are ([kg/m^3]):

- Fresh water: $\rho = 1000$
- Salt water: $\rho = 1025$
- Brackish water: $\rho = 1013$

B.1.2 Specific weights

The used values for the specific weights are ([kN/m^3]):

- Water: $\gamma_w = 10$
- Concrete: $\gamma_{concrete} = 24$
- Soil (sand, saturated): $\gamma_{soil} = 20$

B.1.3 Gravitational acceleration

The used value for the gravitational acceleration is 9.81 m/s^2.

B.1.4 Kinematic viscosity

The kinematic viscosity is assumed to be equal to $1.33 \cdot 10^{-6}$ m^2/s. This is the value of the kinematic viscosity for water at 10°C [3].

B.1.5 Roughness

The roughness for the concrete tunnel is assumed to be equal to $3.5 \cdot 10^{-4}$ m [4].

B.1.6 Safety factors

The used values for the safety factors are ([-]):

- Positive working: $\gamma_{g\downarrow} = 0.9$
- Negative working: $\gamma_{g\uparrow} = 1.2$

B.1.7 Water depths

Two assumptions are made for the determination of the water depths:

1. The fresh water intake should be able to take water in at average low water (ALW).
2. The brackish water outfall should be able to discharge water at average high water (AHW).

The used water depths are given in table B.1:

	Haringvliet	Lake Grevelingen	North Sea
Bottom level	NAP – 6.00 m	NAP – 6.30 m	NAP – 5.60 m
ALW	NAP + 0.50 m	No tide	NAP – 0.80 m
AHW	NAP + 0.70 m	No tide	NAP + 1.80 m
Used water depth	6.50 m	6.30 m	7.40 m

Table B.1: The used water depths.

B.1.8 Height soil layer

The height of the soil layer on top of the intake tunnel is assumed to be equal to 1 m (see section 0).

B.1.9 Maximum span length

The maximum span length of the concrete intake tunnel is assumed to be equal to 6 m (see section 0).

B.1.10 Blockage percentage intake tower

The blockage percentage of the intake tower, the area that is blocked by concrete supports and filter screens, is assumed to be equal to 10%.

B.1.11 Unit price intake tunnel

The unit price of the intake tunnel is 500 €/m^3 [5]. This value is based on the costs of a tunnel and a culvert.

B.1.12 Unit price intake tower

The capital costs of the intake tower are assumed to be negligible compared to the capital costs of the intake tunnel. This is because the length of the tunnel is much larger than the height of the intake tower.

B.2 Basic design

B.2.1 Intake tower

For the basic design of the intake tower, some assumptions are made:

1. The diameter of the intake tower is equal to the width of the intake tunnel.
2. A layer of 1m sand is placed on top of the intake tunnel for counterbalance.
3. The height of the flow area runs from 1 m above the top of the sand layer till 1 m below the average water level.
4. The blockage percentage of the flow area is 10%.
5. The height of the emerged part of the tower is equal to the water depth.

The basic design of the intake tower is given in figure B.1:

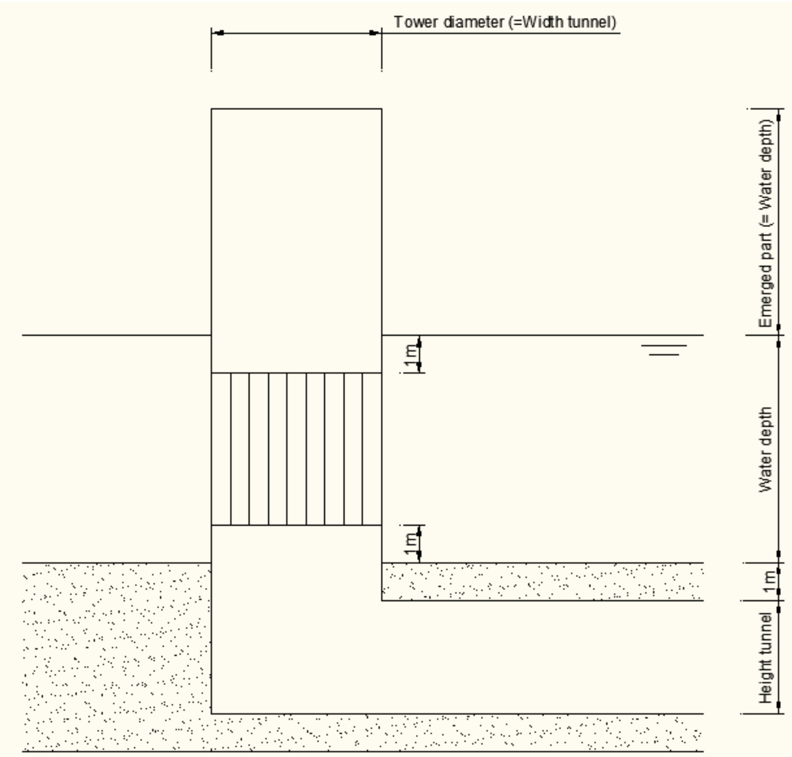


Figure B.1: Basic design intake tower.

The total flow area of the intake is given by:

$$A_{flow} = 0.9\pi \cdot w_{intake} \cdot \left(d_{water} - 2\right) \quad \textbf{(B.1)}$$

The intake velocity is given by:

$$u_{intake} = \frac{Q_{intake}}{A_{flow}} \quad \textbf{(B.2)}$$

B.2.2 Intake tunnel

The basic design of the intake tunnel is given in figure B.2:

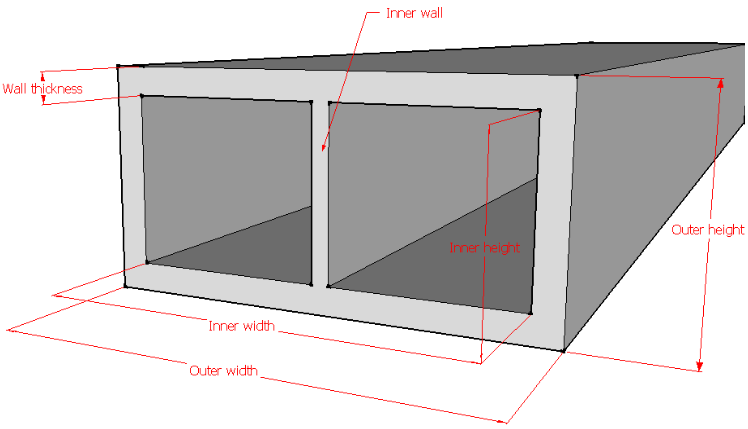


Figure B.2: Basic design intake tunnel.

The inner height and width of the intake tunnel will be obtained by the value of the equivalent diameter. The required wall thickness is the maximum of the following two criteria:

1. The required thickness to bear the weight of the soil layer and water in the case when the intake tunnel is empty (e.g. during maintenance).
2. The required thickness to create a sufficient weight in order to prevent buoyancy.

For simplicity it is assumed that the walls, bottom slab and upper slab all have the same thickness.

1. <u>Bear the weight of the soil layer and water</u>

The required thickness to bear the weight of the soil layer and water can be calculated accurately. In the case study however, a rule of thumb (R.O.T.) is used [6]:

$$d_{R.O.T.} = \frac{L_{span}}{15} \qquad \textbf{(B.3)}$$

According to the rule of thumb, the length of the span can grow infinitely which is a bit unrealistic. For large spans one or more internal support walls are required. In the case study a maximum span length (L_{max}) of 6 m is taken. The number of support walls and the length of the span can be calculated by:

$$n_{\sup} = round\,up\left(\frac{w_{inner}}{L_{\max}} - 1\right)$$
$$L_{span} = \frac{w_{inner}}{\left(n_{\sup}+1\right)} \quad \textbf{(B.4)}$$

When the number of support walls and span length are known, the required wall thickness can be calculated according to equation B.3.

The outer dimensions of the intake tunnel are:

$$w_{outer} = w_{inner} + 2d + n_{\sup} d$$
$$H_{outer} = H_{inner} + 2d \quad \textbf{(B.5)}$$

2. Prevent buoyancy

Another requirement is that the mass of an empty intake tunnel plus the mass of the soil layer, multiplied with a 'positive working' safety factor, should be larger than the mass of the displaced water, multiplied with a 'negative working' safety factor. In this way, the intake tunnel won't become buoyant when the intake tunnel is empty during maintenance. The total mass working positive is given by (for simplicity the subscripts inner, outer, concrete and soil are replaced by respectively i, o, c and s):

$$\left[(w_o \cdot H_o) - (w_i \cdot H_i)\right] \cdot \gamma_c \cdot \gamma_{g\downarrow} + w_o \cdot H_s \cdot \gamma_s \cdot \gamma_{g\downarrow}$$

$$\left[\left(w_i + 2d + n_{\sup} d\right) \cdot \left(H_i + 2d\right) - (w_i \cdot H_i)\right] \cdot \gamma_c \cdot \gamma_{g\downarrow} + (w_i + 2d + n_{\sup} d) \cdot H_s \cdot \gamma_s \cdot \gamma_{g\downarrow} \quad \textbf{(B.6)}$$

$$\left[4d^2\left(1 + \frac{n_{\sup}}{2}\right) + 2d\left(w_i + h_i\left(1 + \frac{n_{\sup}}{2}\right) + \left(H_s\left(1 + \frac{n_{\sup}}{2}\right) \cdot \frac{\gamma_s \cdot \gamma_{g\downarrow}}{\gamma_c \cdot \gamma_{g\downarrow}}\right)\right)\right] \cdot \gamma_c \cdot \gamma_{g\downarrow} + w_i \cdot H_s \cdot \gamma_s \cdot \gamma_{g\downarrow}$$

The mass of the displaced water (working negative) is given by:

$$\left(w_o \cdot H_o\right) \cdot \gamma_w \cdot \gamma_{g\uparrow}$$

$$\left[\left(w_i + 2d + n_{\sup} d\right) \cdot \left(H_i + 2d\right)\right] \cdot \gamma_w \cdot \gamma_{g\uparrow} \quad \textbf{(B.7)}$$

$$\left[4d^2\left(1 + \frac{n_{\sup}}{2}\right) + 2d\left(w_i + H_i\left(1 + \frac{n_{\sup}}{2}\right)\right) + w_i \cdot H_i\right] \cdot \gamma_w \cdot \gamma_{g\uparrow}$$

The mass of the tunnel and soil must be larger than the mass of the displaced water, so:

$$m_{tunnel+soil} > m_{displaced\ water}$$
$$m_{tunnel+soil} - m_{displaced\ water} > 0$$

$$4d^2\left(1+\frac{n_{\sup}}{2}\right)\gamma + 2d\left[\left(w_i + H_i\left(1+\frac{n_{\sup}}{2}\right)\right)\gamma + H_s\left(1+\frac{n_{\sup}}{2}\right)\cdot\gamma_s\cdot\gamma_{g\downarrow}\right] + w_i\cdot H_s\cdot\gamma_s\cdot\gamma_{g\downarrow} - w_i\cdot H_i\cdot\gamma_w\cdot\gamma_{g\uparrow} > 0 \quad \textbf{(B.8)}$$

$$\gamma = \left(\gamma_c\cdot\gamma_{g\downarrow} - \gamma_w\cdot\gamma_{g\uparrow}\right)$$

The expression of equation B.8 is a quadratic equation which can be solved by:

$$d_{buoy} = \frac{-b+\sqrt{b^2-4\cdot a\cdot c}}{2a}$$

$$a = 4\left(1+\frac{n_{\sup}}{2}\right)\gamma \quad \textbf{(B.9)}$$
$$b = 2\left[\left(w_i + H_i\left(1+\frac{n_{\sup}}{2}\right)\right)\gamma + H_s\left(1+\frac{n_{\sup}}{2}\right)\cdot\gamma_s\cdot\gamma_{g\downarrow}\right]$$
$$c = w_i\cdot H_s\cdot\gamma_s\cdot\gamma_{g\downarrow} - w_i\cdot H_i\cdot\gamma_w\cdot\gamma_{g\uparrow}$$

3. Used wall thickness

The used wall thickness is the maximum of the two values, rounded up to a usable wall thickness:

$$d = \max\begin{cases} d_{R.O.T} \\ d_{buoy} \end{cases} \quad \textbf{(B.10)}$$

B.3 Energy level at the end of the power plant

The discharge of brackish water is a special case because the discharge is influenced by the North Sea tide. The average high water is NAP + 1.8 m, but during a storm surge the outside water level can reach levels up to NAP + 3.6m [1]. These levels ensure that the power plant, with a surface level of NAP + 1.0 m, is not able to discharge the brackish water. This problem can be solved by:

- Constructing a temporary storage, discharge the brackish water during low water.
- Operating the power plant at an optimum end energy level and stop the production process in cases when the optimum end energy level is exceeded.
- Operating the power plant at surface level and pump the brackish water out of the power plant in cases when the optimum end energy level is exceeded.

The first solution will not be considered because a temporary storage for the brackish water discharge rate becomes too large.

The optimum end energy level for the power plant is determined as follows:

1. The power plant end level varies between surface level (NAP +1.0 m) and storm surge level (NAP +3.6 m).
2. For every power plant end level the annual initial energy loss ([GWh/yr]) is calculated. The initial energy loss is the extra energy required to pump up the fresh and salt water compared to surface level:

$$E_{loss1} = \rho \cdot g \cdot \left(Q_{fresh} + Q_{salt}\right) \cdot H \cdot t_{hrs/yr} \tag{B.11}$$

3. The discharge of the brackish water causes a total head loss. It is assumed that the maximum allowed head loss is equal to 1 m. This means that the maximum water level where the power plant is still able to discharge is equal to the power plant end level minus the maximum allowed head loss.
4. Using data [1], the percentages are determined in which the water levels of step 3 are exceeded. The percentage of exceeding represents the downtime of the power plant.
5. The energy loss because of the power plant downtime are calculated:

$$E_{loss2} = P_{power\ plant} \cdot t_{hrs/yr} \cdot \%_{downtime} \tag{B.12}$$

6. The total energy loss is calculated for each power plant end level.
7. The optimum power plant end level is achieved by minimizing the total energy loss.

The results of step 1 – 7 are given in table B.2:

Power plant end level		Max. head loss	Max. water level	$E_{loss;1}$	Downtime	$E_{loss;2}$	$E_{loss;tot}$
		[m]		[GWh/yr]	[%]	[GWh/yr]	[GWh/yr]
NAP +1.0m	SL	1	NAP +0.0m	0.00	43.40%	95.11	95.11
NAP +1.1m		1	NAP +0.1m	0.54	40.50%	88.76	89.30
NAP +1.2m		1	NAP +0.2m	1.08	37.86%	82.97	84.05
NAP +1.3m		1	NAP +0.3m	1.62	35.23%	77.21	78.82
NAP +1.4m		1	NAP +0.4m	2.16	32.51%	71.25	73.40
NAP +1.5m		1	NAP +0.5m	2.70	29.61%	64.89	67.59
NAP +1.6m		1	NAP +0.6m	3.24	26.69%	58.49	61.73
NAP +1.7m		1	NAP +0.7m	3.78	23.65%	51.83	55.61
NAP +1.8m	AHW	1	NAP +0.8m	4.32	20.45%	44.82	49.13
NAP +1.9m		1	NAP +0.9m	4.85	17.27%	37.85	42.70
NAP +2.0m		1	NAP +1.0m	5.39	14.00%	30.68	36.08
NAP +2.1m		1	NAP +1.1m	5.93	10.77%	23.60	29.54
NAP +2.2m		1	NAP +1.2m	6.47	8.11%	17.77	24.25
NAP +2.3m		1	NAP +1.3m	7.01	5.89%	12.91	19.92
NAP +2.4m		1	NAP +1.4m	7.55	4.12%	9.03	16.58
NAP +2.5m		1	NAP +1.5m	8.09	2.82%	6.18	14.27
NAP +2.6m		1	NAP +1.6m	8.63	1.90%	4.16	12.79
NAP +2.7m		1	NAP +1.7m	9.17	1.28%	2.81	11.98

NAP +2.8m		1	NAP +1.8m	9.71	0.86%	1.88	11.59
NAP +2.9m		1	NAP +1.9m	10.25	0.60%	1.31	11.56
NAP +3.0m		1	NAP +2.0m	10.79	0.42%	0.92	11.71
NAP +3.1m		1	NAP +2.1m	11.33	0.30%	0.66	11.99
NAP +3.2m		1	NAP +2.2m	11.87	0.22%	0.48	12.35
NAP +3.3m		1	NAP +2.3m	12.41	0.16%	0.35	12.76
NAP +3.4m		1	NAP +2.4m	12.95	0.11%	0.24	13.19
NAP +3.5m		1	NAP +2.5m	13.49	0.07%	0.15	13.64
NAP +3.6m	SSL	1	NAP +2.6m	14.03	0.05%	0.11	14.14

Table B.2: The determination of the optimum power plant end level.

From table B.2 can be concluded that the optimum level at the end of the production process is NAP +2.9m. The annual energy loss because of the extra pumping compared to surface level and power plant downtime is 11.56 GWh/yr.

B.4 Theory

B.4.1 General

The transport of water from the intake tower to the pump pit is a pressurized transport. The driving force behind this kind of transport is a difference in energy level, represented by a water level difference, on each side of the intake tunnel. This difference will cause a flow from the side with the high energy level to the side with the low energy level. The energy level of a water column is a summation of three hydraulic heads:

- Static head $z_{1,2}$
- Pressure head $\dfrac{p_{1,2}}{\rho g}$
- Velocity head $\dfrac{u_{1,2}^2}{2g}$

During the transport energy is lost due to friction and local losses. Local losses are caused by sudden changes in cross section or direction. The water transport stops when the energy level (the summation of hydraulic heads and losses) on the low energy level side is equal to the energy level on the high energy level side.

The abovementioned transport can be described by the Bernoulli formula:

$$z_1 + \frac{p_1}{\rho_1 g} + \frac{u_1^{\,2}}{2g} = z_2 + \frac{p_2}{\rho_2 g} + \frac{u_2^{\,2}}{2g} + h_f + h_L \qquad \textbf{(B.13)}$$

When the water source at the high energy level side is assumed to be infinite and the axis of the tunnel is taken as reference level, the Bernoulli equation reduces to:

$$z_1 = \frac{p_2}{\rho_2 g} + \frac{u_2^{\ 2}}{2g} + h_f + h_L \qquad \textbf{(B.14)}$$

The reduced Bernoulli equation is visualized in figure B.3:

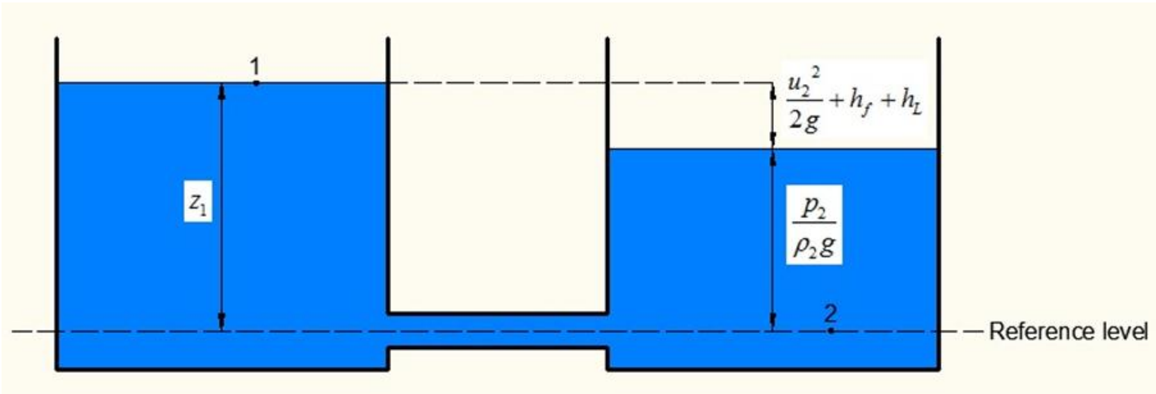


Figure B.3: Schematization of the pressurized water transport.

B.4.2 Velocity head

The head loss due to the velocity, the velocity head, depends on the design flow rate and the inner cross section of the intake tunnel:

$$h_{velocity} = \frac{u^2}{2g} = \frac{8Q^2}{g\pi^2 D_{eq}^4} \qquad \textbf{(B.15)}$$

Equation B.15 states that the head loss due to the velocity depends on the cross section of the intake tunnel. The basic design of the intake tunnel implies a rectangular inner cross section. However, in equation B.15 a diameter is used which implies a circular inner cross section. The diameter in equation B.15 is called the equivalent diameter. The equivalent diameter is the diameter of a circular duct or pipe that gives the same pressure loss as a rectangular duct or pipe.

B.4.3 Friction loss

The head loss due to friction can be calculated by the Darcy-Weissbach formula:

$$h_f = f\,\frac{L}{D_{eq}}\frac{u^2}{2g} = f\,\frac{L}{D_{eq}}\frac{8Q}{g\pi^2 D_{eq}^4} \qquad \textbf{(B.16)}$$

Equation B.16 states that the head loss due to friction depends on the length of the tunnel, the inner cross section and a friction coefficient. The magnitude of the friction coefficient depends on the Reynolds number. The Reynolds number is a dimensionless parameter which quantifies the flow regime in the intake tunnel:

$$\mathrm{Re} = \frac{uD_{eq}}{v} \tag{B.17}$$

The flow regime can be laminar or turbulent:

- Laminar for Re < 2000
- Turbulent for Re > 4000

For 2000 < Re < 4000, the flow is in a transition region.

The pressurized transport of water through the intake tunnel is always turbulent because the velocity through the intake tunnel and equivalent diameter will always result in a high order Reynolds number:

$$\frac{uD_{eq}}{v} >> 4000 \tag{B.18}$$

The friction coefficient for a turbulent flow is determined iteratively by the Colebrook expression:

$$\frac{1}{\sqrt{f}} = -2\log\left(0.27\frac{k_N}{D_{eq}} + \frac{2.5}{\mathrm{Re}\sqrt{f}}\right) \tag{B.19}$$

The drawback of iteration using equation B.19 was circumvented by Jain, who suggested the following explicit equation for the friction factor [7]:

$$\frac{1}{\sqrt{f}} = -2\log\left(0.27\frac{k_N}{D_{eq}} + \frac{5.74}{\mathrm{Re}^{0.9}}\right) \qquad or \qquad f = \frac{0.25}{\left[\log_{10}\left(0.27\frac{k_N}{D_{eq}} + \frac{5.74}{\mathrm{Re}^{0.9}}\right)\right]^2} \tag{B.20}$$

$$for: \qquad 10^{-6} \le \frac{k_N}{D_{eq}} \le 10^{-2} \qquad and \qquad 5000 \le \mathrm{Re} \le 10^8$$

B.4.4 Local losses

Local energy losses depend on the velocity in the intake tunnel and can therefore be expressed in an analogue formula as the Darcy-Weissbach equation:

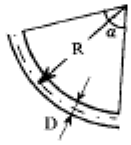
$$h_L = \sum \xi \frac{u^2}{2g} = \sum \xi \frac{8Q^2}{g\pi^2 D_{eq}^4} \tag{B.21}$$

Equation B.21 states that the head loss due to local losses depends on the inner cross section of the intake tunnel and the summation of loss factors. The loss factors can be found in engineering handbooks. The factors used in the case study are [8]:

Friction Coefficient K_w for Flow through Tube Systems

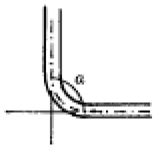
Reynolds for downstream conditions $Re > 10^5$

smooth bent



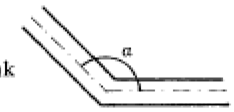
$$K_w = \{0.163 \left(\frac{D}{R}\right)^{3.5} + 0.131\} \frac{\alpha}{90}$$

sharp bent



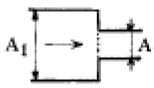
α	90	120	135	150	165
K_w	1.30	0.50	0.26	0.11	0.02

kink



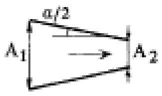
α	40	60	80	90	100	120	140	160
K_w	2.43	1.86	1.26	0.98	0.74	0.36	0.14	0.05

sudden contraction



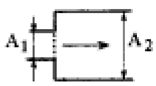
$$K_w = 0.45 \{1 - \frac{A_2}{A_1}\}$$

gradual contraction



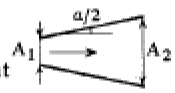
α	10	20	30	40	50	60	70	80
K_w	0.16	0.20	0.24	0.28	0.31	0.32	0.34	0.35

sudden enlargement



$$K_w = \{\frac{A_2}{A_1} - 1\}^2$$

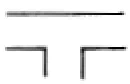
gradual enlargement



$$K_w = K \{1 - \frac{A_1}{A_2}\}^2$$

α	<10	10	20	30	40	50	60	70	80	90
K	0	0.17	0.41	0.71	0.90	1.03	1.12	1.13	1.10	1.05

T-junction



K_w	1,3	1,5	1,0	0,4

bents

	45° smooth	90° smooth	90° sharp	180°
K_w	0,35	0,75	1,3	1,5

pipe entrance

	sharp	rounded
K_w	0,5	0,05

pipe exit

	free yet	confined yet
K_w	0	1,0

Figure B.4: Loss factors.

B.4.5 Total head loss

The total head loss can now be described by using equation B.23:

$$z_1 - \frac{p_2}{\rho_2 g} = \frac{u^2}{2g} + h_f + h_L$$

$$z_1 - \frac{p_2}{\rho_2 g} = \frac{8Q^2}{g\pi^2 D_{eq}^4} + f\frac{L}{D_e}\frac{8Q^2}{g\pi^2 D_{eq}^4} + \sum\xi\frac{8Q^2}{g\pi^2 D_{eq}^4} \qquad \textbf{(B.22)}$$

$$\Delta h = \frac{8Q^2}{g\pi^2 D_{eq}^4}\left(1 + f\frac{L}{D_{eq}} + \sum\xi\right)$$

B.5 Cross section analysis

The total head loss given in equation B.22 will be used to determine the required cross section of the intake tunnel. The last expression of equation B.22 has a number of unknown parameters:

- The flow rate
- The equivalent diameter
- The length of the tunnel
- The summation of the loss factors
- The friction coefficient

The required equivalent diameter has a major impact on the total head loss. According to equation B.22, a twice as large equivalent diameter results in a 16 times smaller head loss when the flow rate is taken as a constant. However, a twice as large equivalent diameter results in larger capital costs. An optimum equivalent diameter should be found by imposing a requirement. In the case studies, the imposed requirement is that the total head loss should not be larger than 1 m.

The optimum equivalent diameter will be determined through iteration for a number of flow rates. The flow rate through the tunnels depends on the power plant capacity and the practical osmotic energy (see section A.4) and is thus a known value. The equivalent diameter results eventually in the outer cross section of the tunnel and the capital costs. The determination of the outer cross section and the capital costs are described stepwise in the following sections.

B.5.1 Power plant location within the given area

The first step is to determine the power plant location within the area given in figure B.5:

Figure B.5: Power plant area.

The fixed trajectory of the intake and outfall systems is already determined in section 6.3.3. The fresh water intake runs from the intake location to the eastern corner of the power plant area given in figure B.5. The fixed length is 2.250 m and the trajectory contains a 90° and a 140° bend. The salt water intake runs from the intake location in a straight line to the middle of the western – southern line. The fixed length is 4.000 m. The brackish outfall runs from the eastern corner of the power plant area to the outfall location. The fixed length is 2.000 m and the trajectory contains a 140° bend. The trajectories are given in figure B.6 and B.7.

Figure B.6: Fresh (cyan) and brackish water (green) tunnel trajectory.

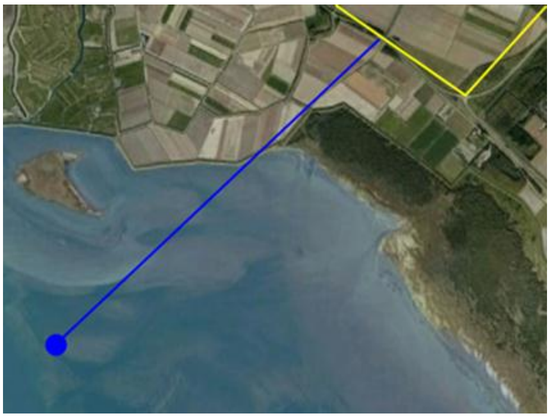


Figure B.7: Salt water tunnel trajectory.

The location of the power plant within the area of figure B.5 depends on the power plant capacity. The higher the power plant capacity, the larger is the required area. For a small capacity power plant, the location of the power plant should be as close as possible to the eastern corner where the fresh water intake and brackish water outfall enter the power plant area. When the power plant capacity increases, the power plant area increases from the eastern corner along the eastern – southern line and the northern eastern line. This implies that the length and trajectory of the fresh water intake and brackish water outfall are constant independent to the power plant capacity. The length of the salt water intake decreases with an increasing power plant capacity.

B.5.2 Tunnel length and local losses

B.5.2.1 Fresh water intake

The trajectory of the fresh water intake is independent to the power plant capacity. The length is 2.250 m. The trajectory consists of a 90° and 140° bend. According to figure B.4, the loss factor of a 90° bend is 1.30. The loss factor could be reduced by applying two bends of 135° instead of one 90° bend. This reduces the loss factor to 0.52. The 140° bend represents a loss factor equal to 0.21 (interpolation between the 135° and 150° bend). Other loss factors are due to the tunnel entrance and exit. It is assumed that the tunnel entrance is smooth, which will decrease the loss factor. The loss factor associated with a smooth tunnel entrance is 0.05. The tunnel ends in a pump sump, so the flow which leaves the tunnel can be considered as a confined yet. The loss factor associated with a confined yet is equal to 1.0. The total loss factor of the fresh water intake tunnel is equal to 1.78. The total loss factor is also independent to the power plant capacity.

B.5.2.2 Salt water intake

The trajectory of the salt water intake depends on the power plant capacity. The higher the power plant capacity, the smaller the length of the salt water intake will be. This is because a larger power plant capacity implies a larger power plant area. Especially the pre-treatment facility appears to have a large influence on the power plant area and thus on the length of the salt water intake (see section 6.4). The length of the salt water intake for a number of fresh water flow rates is given in table. These lengths were determined after the design of the pre-treatment facility.

Salt water flow rate [m^3/s]	**Salt water intake length [m]**
0	6.200
2	6.185
4	6.175
6	6.165
8	6.155
10	6.145
20	6.095
30	6.035
40	5.985
50	5.930
60	5.876
70	5.822
80	5.768
90	5.714
100	5.660
150	5.390
200	5.300
250	5.210
300	5.120
350	4.940
400	4.760
500	4.400

Table B.3: Length salt water intake for a number of fresh water flow rates.

The trajectory of the salt water intake contains a 150° bend, which has a loss factor of 0.11. The total loss factor is, together with the tunnel entrance and exit loss factors, equal to 1.16. The total loss factor is independent to the power plant capacity.

B.5.2.3 Brackish water outfall

The tunnel length and total loss factor of the brackish water outfall are independent to the power plant capacity. The length of is equal to 2.000 m, and the total loss factor of the brackish water outfall is equal to 1.26.

B.5.3 Required equivalent diameter

The required equivalent diameter will be determined through iteration. The iteration steps are given below:

1. A certain flow rate is considered.
2. For a varying equivalent diameter the cross section, velocity, Reynolds number (equation B.17) and friction factor (equation B.20) are determined.
3. For a varying equivalent diameter, the total head loss is determined.
4. The equivalent diameter which corresponds to a head loss of 1 m is the optimum equivalent diameter corresponding with the considered flow rate in step 1.
5. Steps 1 – 4 are executed multiple times for a number of flow rates.

The results of the iteration are given in figure B.8 – B.10:

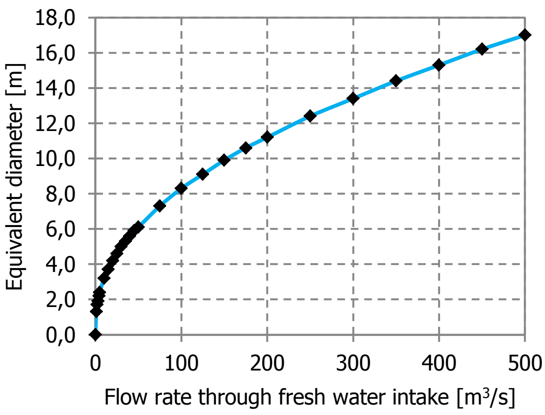


Figure B.8: Required equivalent diameter for the fresh water intake tunnel.

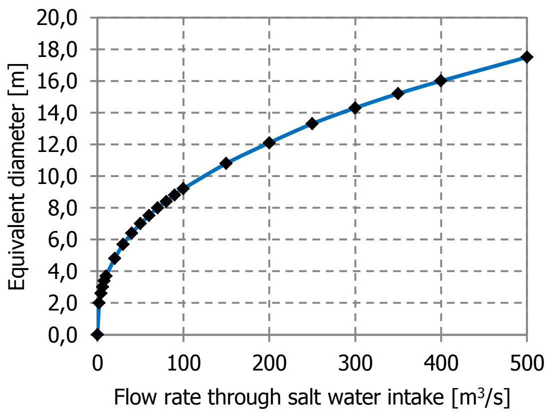


Figure B.9: Required equivalent diameter for the salt water intake tunnel.

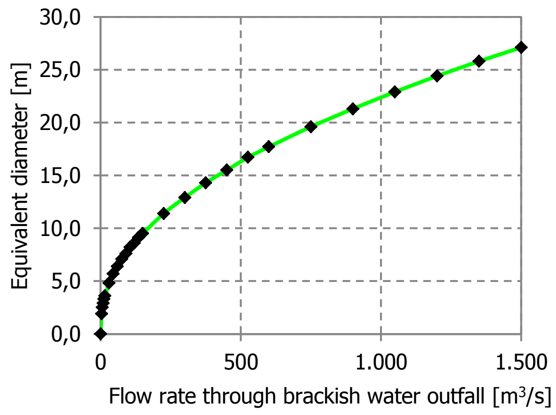


Figure B.10: Required equivalent diameter for the brackish water outfall.

The required equivalent diameters in case of the 25 MW PRO power plant are:

- Fresh water intake: $D_{eq} = 4.1m$
- Salt water intake: $D_{eq} = 6.5m$
- Brackish water outfall: $D_{eq} = 6.4m$

B.5.4 Required inner cross section

The next step is to obtain the required inner cross section:

$$A_{inner} = 0.25\pi \cdot D_{eq}^2 \quad \textbf{(B.23)}$$

The results are given in figure B.11 – B.13:

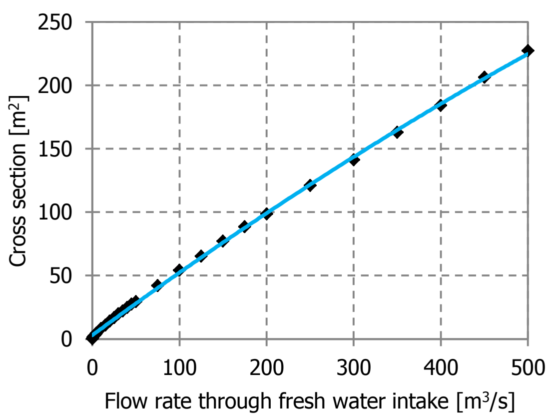


Figure B.11: Required inner cross section for the fresh water intake tunnel.

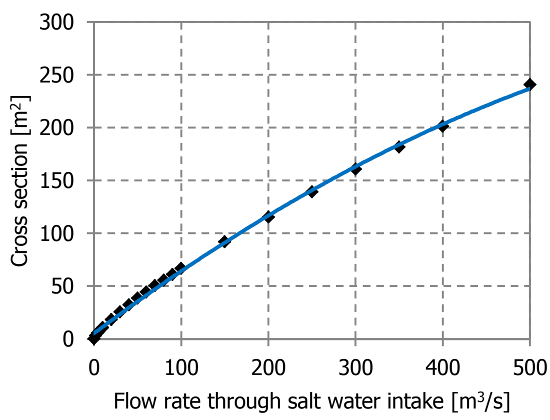


Figure B.12: Required inner cross section for the salt water intake tunnel.

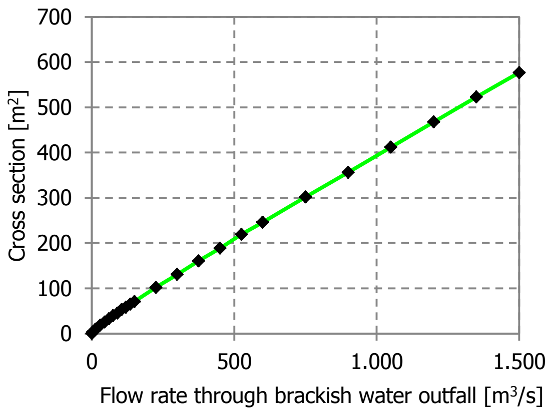


Figure B.13: Required inner cross section for the brackish water outfall.

The required inner cross sections in case of the 25 MW PRO power plant are:

- Fresh water intake: $A_{inner} = 13.2\,m^2$
- Salt water intake: $A_{inner} = 33.3\,m^2$
- Brackish water outfall: $A_{inner} = 32.1\,m^2$

B.5.5 Required outer cross section

The next step is to determine the required outer cross section of the tunnels. In order to determine the outer cross section, first the amount of internal support walls and wall thickness should be determined. This determination is described in section 0. Because the height of the tunnels cannot increase infinitely, a height of 3 m is taken as a constant.

The number of internal supports and the wall thickness result in a required outer cross section. The results of the determination are given in figure B.14 – B.16:

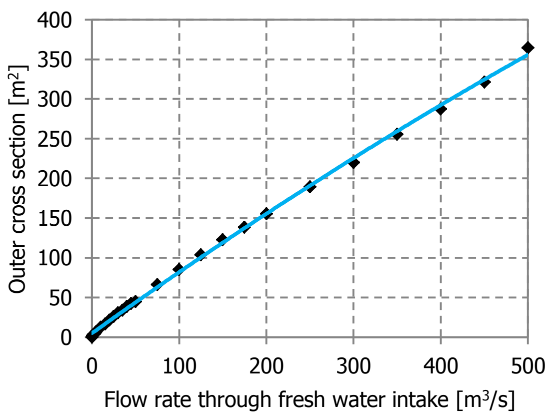


Figure B.14: Required outer cross section for the fresh water intake tunnel.

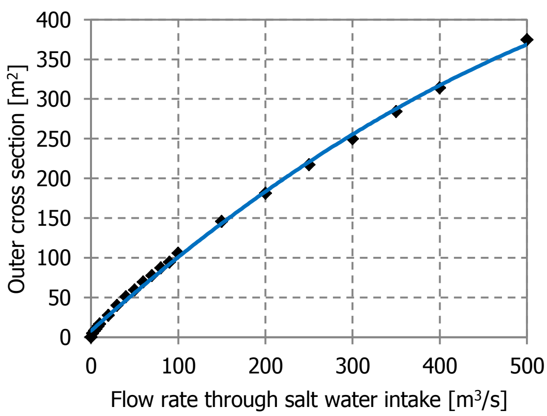


Figure B.15: Required outer cross section for the salt water intake tunnel.

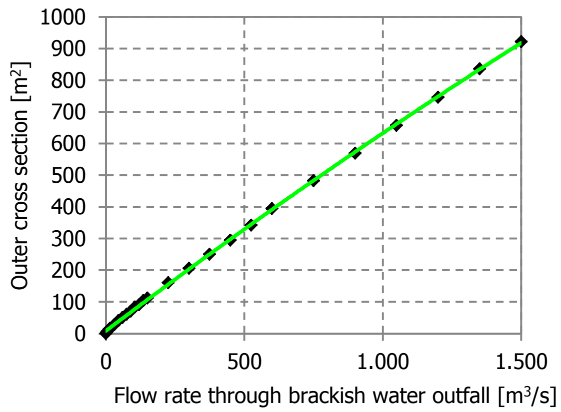


Figure B.16: Required outer cross section for the brackish water outfall.

The required outer cross sections in case of the 25 MW PRO power plant are:

- Fresh water intake: $n_{sup} = 0$, $d_{wall} = 0.45\,m$, $A_{outer} = 20.7\,m^2$
- Salt water intake: $n_{sup} = 1$, $d_{wall} = 0.55\,m$, $A_{outer} = 52.1\,m^2$
- Brackish water outfall: $n_{sup} = 1$, $d_{wall} = 0.55\,m$, $A_{outer} = 50.8\,m^2$

B.5.6 Capital costs per tunnel

The outer cross section of the tunnel, the tunnel length and the unit rate of a tunnel result in the capital costs. The results are given in figure B.17 – B.19:

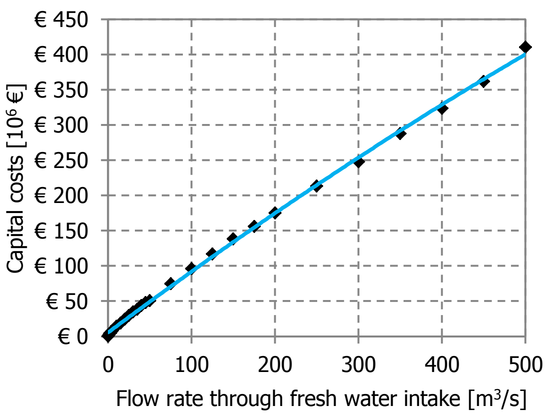


Figure B.17: Capital costs fresh water intake tunnel.

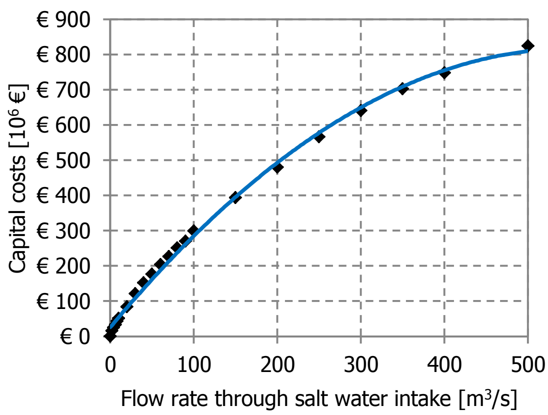


Figure B.18: Capital costs salt water intake tunnel.

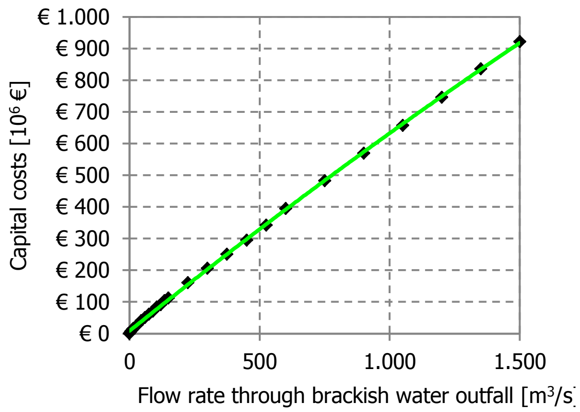


Figure B.19: Capital costs brackish water outfall.

The capital costs in case of the 25 MW PRO power plant are:

- Fresh water intake: $C_{fresh\ intake} = A_{outer} \cdot L_{tunnel} \cdot C_{€/m^3} = €23.287.500,-$
- Salt water intake: $C_{salt\ intake} = A_{outer} \cdot L_{tunnel} \cdot C_{€/m^3} = €155.648.750,-$
- Brackish water outfall: $C_{tunnel} = A_{outer} \cdot L_{tunnel} \cdot C_{€/m^3} = €50.800.000,-$

B.5.7 Total capital costs per power plant capacity

B.5.7.1 PRO

The total capital costs for the intake and outfall systems in the case of a PRO power plant are obtained by a summation of the capital costs of the fresh water intake, salt water intake and brackish water outfall. The ratio between the flow rates in case of a PRO power plant are about 1:2:3. With a practical osmotic energy of about 1.20 MJ/m^3, the relation between power plant capacity and total capital costs of the intake and outfall systems is:

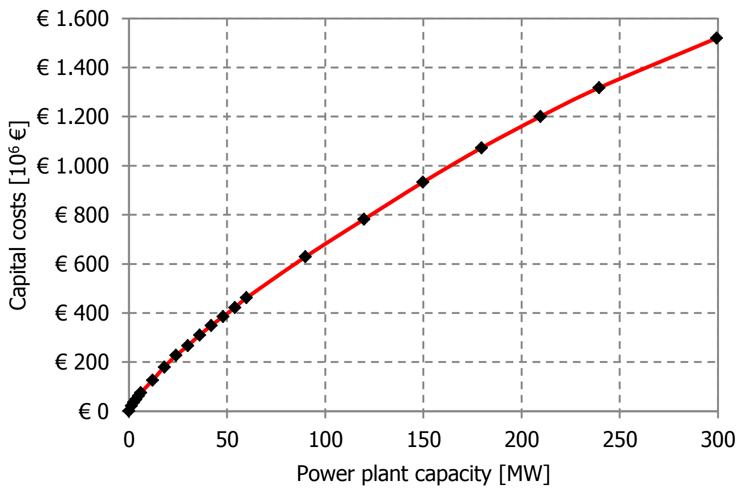


Figure B.20: Total capital cost intake and outfall systems in case of a PRO power plant.

The total capital costs of the intake and outfall systems in case of the 25 MW PRO power plant are €230.000.000,-.

B.5.7.2 RED

The same calculation can be executed in the case of a RED power plant. The difference between PRO and RED is that the practical energy in the case of RED is lower. This will increase the required flow rates through the power plant and thus the capital costs. Another difference is the ratio between fresh and salt water through the power plant. This ratio is in the case of a RED 1:1. This will reduce the capital costs of the salt water intake tunnel. The relation between power plant capacity and total capital costs of the intake and outfall systems in the case of a RED power plant is given in figure B.21:

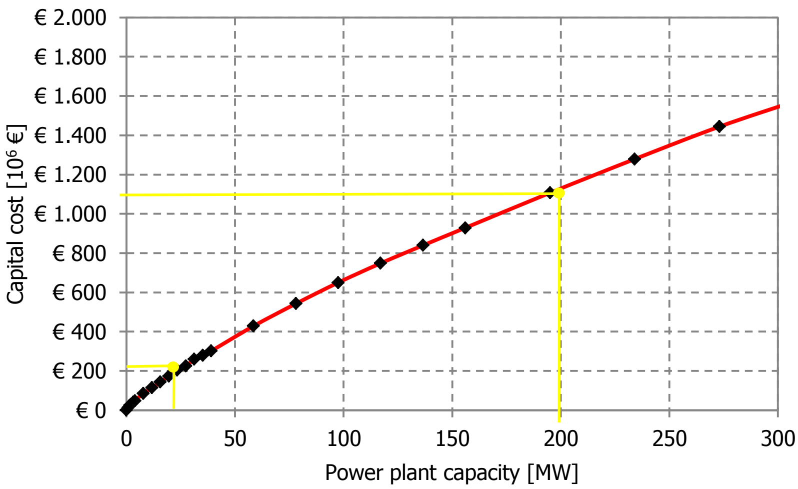


Figure B.21: Total capital cost intake and outfall systems in case of a RED power plant.

Appendix C
Pre-treatment

C.1 Types of treatment

Contaminated water contains the following substances – presented in decreasing particle size:

- Suspended solids
- Colloidal particles
- Dissolved solids
- Micro-organisms

These substances can be removed from the water by using the following treatments.

C.1.1 Coagulation

Colloidal particles are negatively charged and are stable in water: these particles are, unlike suspended solids, not self-settling. By adding a certain substance, a so-called coagulant, the charge of the colloidal particles is neutralised. This neutralisation ensures that the colloidal particles coagulate. The coagulated particles will settle or float and can therefore be removed from the water. This process is called coagulation.

C.1.2 Flocculation

The coagulation process described in section C.1.1 can be accelerated by adding flocculants to the water. These flocculants ensure that larger flocs are achieved.

C.1.3 Sedimentation

Sedimentation is a process in which the flocs of section C.1.1 and C.1.2 and other suspended solids settle and can be removed from the water.

C.1.4 Sand filtration

Sand filtration is a process that is used to remove particles and suspended solids that can precipitate on the membranes. The filter consists of multiple layers of sand with different particle size and density. Suspended solids of sizes larger than the particle sizes of the sand filter will be removed from the water.

C.1.5 Granular Activated Carbon filtration

Granular Activated Carbon filtration is a process where dissolved solids can be removed by using a solid (activated carbon). The dissolved solids are removed by absorption. GAC filtration is used to eliminate the nutrients in water for bacteria.

C.1.6 Fine filtration

Filtration is used to remove particles and micro-organisms from water. There are multiple filtration techniques available. Each technique has its own area of application. The techniques are given in figure C.1:

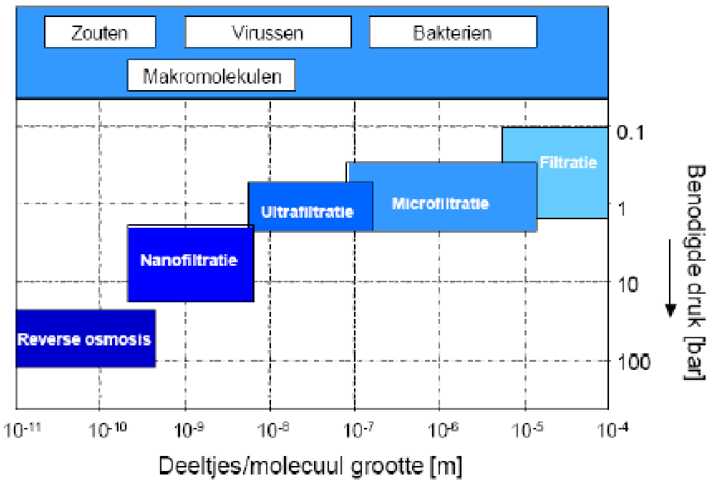


Figure C.1: Overview filtration techniques for different particles and particle size.

C.1.7 pH neutralization

pH neutralisation is a chemical reaction in where an acid and a base form a salt. This process is often applied to reduce the damage that an effluent may cause upon release to the environment.

C.2 Micro-filtration

In the case study, use is made of the information given on the website of the company Hubert [9]. Hubert is a company specialized in water treatment installations.

HUBERT
water treatment installations

Hubert micro screen

Hubert micro screen
The Micro screen is developed for very fine filtration of surface, process and recirculation water.

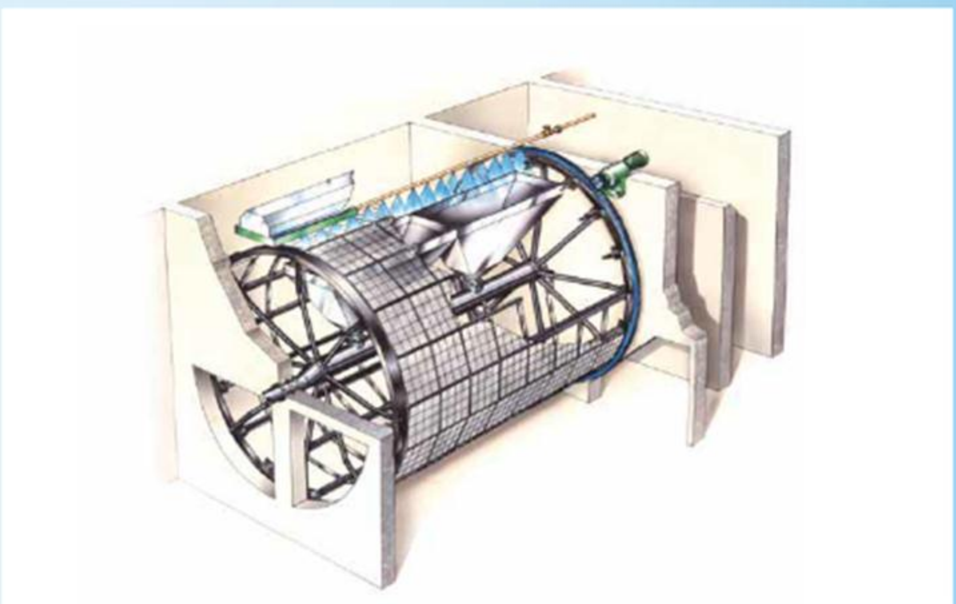
The Micro screen is a rotating drum screen, provided with fine mesh screen panels or a filter cloth.

The dirt particles, which are trapped on the screen panels, will be removed by high pressure spray water and discharged through the trash hoppers and the hollow shaft.
The cleaning process can be executed automatically and semi-automatically.

Application
The Micro screen is suitable for:
- Pre-treatment of waste water
- Pre-treatment of potable water
- Effluent polishing
- Process water filtration (recycling)
- Feed water filtration for desalination plants
- Cooling water filtration

Advantages
The advantages of the Micro screen are:
- Resistant to corrosion
- Very fine filtration possible
- Reliable
- Low maintenance
- Water lubricated bearings
- Environmental friendly
- Long lifetime

Standard dimensions
Capacity: between 40 and 1500 m³/hr.
Mesh opening: between 11 and 500 µm.
Drum length: up to 6 m.
Diameter: between 0,7 and 3,5 m.
Material: stainless steel AISI 304/316/316Ti
Specific dimensions or materials on request.



Kooyweg 20 - PO Box 29, 8715 ZH Stavoren - The Netherlands
T: +31 (0) 514 684 444 - E: info@www.hubert.nl - www.hubert.nl

Figure C.2: Characteristics micro-filtration according to Hubert.

C.3 Calculations

The calculations conducted in this section are the calculations associated with the 25 MW PRO power plant. These calculations are similar to the calculations required for the other power plants.

C.3.1 Characteristics

- Capacity: 1.500 m^3/hr
- Diameter: 3.5 m
- Length: 6.0 m
- Volume: 57.73 m^3
- Filter area: 65.97 m^2

C.3.2 Water flux

The water flux is found by dividing the capacity through the filter area of a drum:

$$J_{w;drum} = \frac{1500}{3600 \cdot 65.97} = 0.0063\,\mathrm{m^3/m^2s} \tag{C.1}$$

C.3.3 Required filter area

The required filter area is found by dividing the flow rate through the water flux:

$$\begin{aligned} A_{filter;\,fresh} &= \frac{Q_{fresh}}{J_{w;drum}} = \frac{20.91}{0.0063} = 3319\,\mathrm{m^2} \\ A_{filter;salt} &= \frac{Q_{salt}}{J_{w;drum}} = \frac{41.82}{0.0063} = 6638\,\mathrm{m^2} \end{aligned} \tag{C.2}$$

C.3.4 Number of drums

The number of drums required is found by dividing the total required filter area through the filter area per drum:

$$\begin{aligned} n_{drum;\,fresh} &= \frac{A_{filter;total}}{A_{filter;drum}} = \frac{3319}{65.97} = 50 \\ n_{drum;salt} &= \frac{A_{filter;total}}{A_{filter;drum}} = \frac{6638}{65.97} = 100 \end{aligned} \tag{C.3}$$

C.3.5 Energy consumption

According to figure C.1, micro-filtration requires a minimum pressure of 0.3 bar. This pressure can be transformed into an energy consumption by substituting this pressure into the equation B.11. For the elevation, a height of 3 m is taken which is the equivalent of 0.3 bar:

$$E_{consumed} = \rho \cdot g \cdot Q \cdot H \cdot t_{hrs/yr}$$

$$E_{consumed;\,fresh} = 5.4\,\text{GWh/yr}$$
$$E_{consumed;salt} = 11.1\,\text{GWh/yr} \tag{C.4}$$

The total consumed energy of the micro-filtration is 16.5 GWh/yr.

C.3.6 Required area

It is assumed that all the drums need a free space of 0.50 m on each side. The required area per is therefore:

$$L_{fresh;drums} = \left(D_{drums} + 2\cdot 0.5\right)\cdot n_{fresh;drums} = 225\,m$$
$$L_{salt;drums} = \left(D_{drums} + 2\cdot 0.5\right)\cdot n_{salt;drums} = 450\,m \tag{C.5}$$

The cross section of the pump sumps is given in figure C.3:

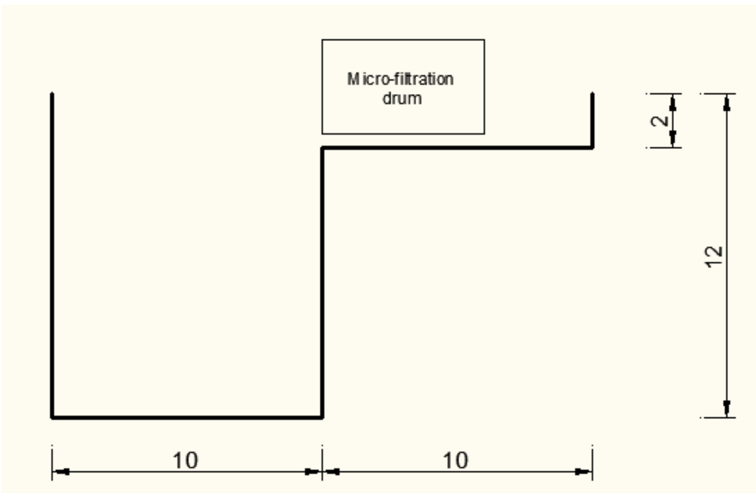


Figure C.3: Cross section of the pump sumps.

The cross section of the discharge sump is given in figure C.4:

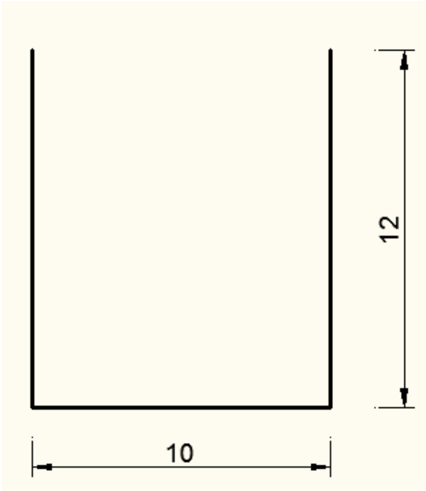


Figure C.4: Cross section of the pump sumps.

The required area is:

$$\begin{aligned} A_{fresh;drums} &= L_{fresh;drums} \cdot w_{pump\,sump} = 4.500\,m^2 \\ A_{salt;drums} &= L_{salt;drums} \cdot w_{pump\,sump} = 9.000\,m^2 \\ A_{discharge\,sump} &= L_{salt;drums} \cdot w_{discharge\,sump} = 4.500\,m^2 \end{aligned} \tag{C.6}$$

C.3.7 Capital costs

The capital costs of the pre-treatment are a function of the annual capacity of the power plant. The unit price for micro-filtration is 0.10 €/m^3.

$$C_{pre-treatment} = Q \cdot t_{sec/yr} \cdot C_{€/m^3} \tag{C.7}$$

The capital costs for the two pre-treatment facilities are:

- Fresh: €65.986.942,-
- Salt: €131.973.883,-

The total capital costs are €197.960.825,-.

The required capital costs of the different sumps area:

$$\begin{aligned} C_{pump\,sumps} &= A_{pump\,sump} \cdot (L_{fresh;drums} + L_{salt;drums}) \cdot C_{€/m^3} = €28.350.000,- \\ C_{discharge\,sump} &= A_{discharge\,sump} \cdot L_{salt;drums} \cdot C_{€/m^3} = €16.200.000,- \end{aligned} \tag{C.8}$$

Appendix D
Membrane stacks

D.1 Calculations

The calculations conducted in this section are the calculations associated with the 25 MW PRO power plant. These calculations are similar to the calculations required for the other power plants.

D.1.1 Characteristics

- Water flux: $2.0 \cdot 10^{-6}$ m/s
- Power density: 2.4 W/m^2
- Packing density: 775 m^2/m^3
- Length module: 1 m
- Diameter module: 0.2 m

D.1.2 Membrane area

The required membrane area depends on the power plant capacity and the power density:

$$A_m = \frac{P_{power\ plant}}{W} = \frac{25 \cdot 10^6}{2.4} = 10.416.667\, m^2 \quad \textbf{(D.1)}$$

D.1.3 Module amount

The amount of modules depends on the required membrane area, the volume of one module and the packing density:

$$n_{modules} = \frac{A_m}{p.d \cdot V_{module}} \quad \textbf{(D.2)}$$

The volume of the modules depends on the length and diameter of the modules:

$$V_{module} = A_{module} \cdot L_{module} = 0.25\pi \cdot D^2_{module} \cdot L_{module} = 0.25\pi \cdot 0.2^2 \cdot 1 = 0.03142\, m^3 \quad \textbf{(D.3)}$$

The amount of modules required is:

$$n_{modules} = \frac{A_m}{p.d \cdot V_{module}} = \frac{10.416.667}{775 \cdot 0.03142} = 427.781 \quad \textbf{(D.4)}$$

D.1.4 Pressure vessel amount

It is assumed that 7 modules are housed in one pressure vessel. The required amount of pressure vessels is therefore:

$$n_{pressure\,vessels} = \frac{n_{modules}}{7} = \frac{427.781}{7} = 61.112 \tag{D.5}$$

D.1.5 Stack amount

It is assumed that the distance between two consecutive pressure vessels is 0.1 m. In the vertical direction, the membrane stacks can house a number of pressure vessels equal to:

$$n_{pressure\,vessels} = \frac{H_{stack}}{D_{pressure\,vessel} + 0.1} = 20 \tag{D.6}$$

In the horizontal direction, the membrane stacks can house a number of pressure vessels equal to:

$$n_{pressure\,vessels} = \frac{L_{stack}}{D_{pressure\,vessel} + 0.1} = 306 \tag{D.7}$$

Each membrane stack can house a total number of 6.120 pressure vessels. The required amount of membrane stacks is therefore:

$$n_{membrane\,stacks} = \frac{n_{pressure\,vessels}}{n_{pressure\,vessels/stack}} = \frac{61112}{6120} = 10 \tag{D.8}$$

In total, 10 membrane stacks are required.

D.1.6 Area per stack

The required area per stack is:

$$A_{membrane\,stack} = L_{membrane\,stack} \cdot L_{pressure\,vessels} = L_{membrane\,stack} \cdot 7 \cdot L_{modules} = 92 \cdot 7 \cdot 1 = 644\,m^2 \tag{D.9}$$

Ten membrane stacks require 6.440 m^2.

D.1.7 Capital costs

The capital costs of the membrane stacks depend on the required membrane area and the unit rate of the membranes:

$$C_{membrane\,stacks} = A_m \cdot C_{€/m^2} = 10416667 \cdot 5 = €52.083.335,- \tag{D.10}$$

The capital costs of the membrane stacks are €52.100.000,-.

D.1.8 Energy loss due to fresh water bleed

The energy loss due to fresh water bleed is determined by using equation D.11:

$$E_{loss;bleed} = \left(1 - \frac{Q_{brackish}}{Q_{fresh}}\right) \cdot E_{initial}$$

$$E_{loss;bleed} = \left(1 - \frac{\frac{1}{3}\left(\left(\left(1 - \%_{bleed}\right)Q_{fresh}\right) + Q_{salt}\right)}{Q_{fresh}}\right) \cdot E_{initial} \qquad \textbf{(D.11)}$$

$$E_{loss;bleed} = \left(1 - \frac{\frac{1}{3}\left(\left(\left(1 - 0.1\right)20.9\right) + 41.8\right)}{20.9}\right) \cdot 204 = 6.8\,GWh/yr$$

Appendix E

Pumps, pipes and turbines

E.1 Mechanical classification of pumps

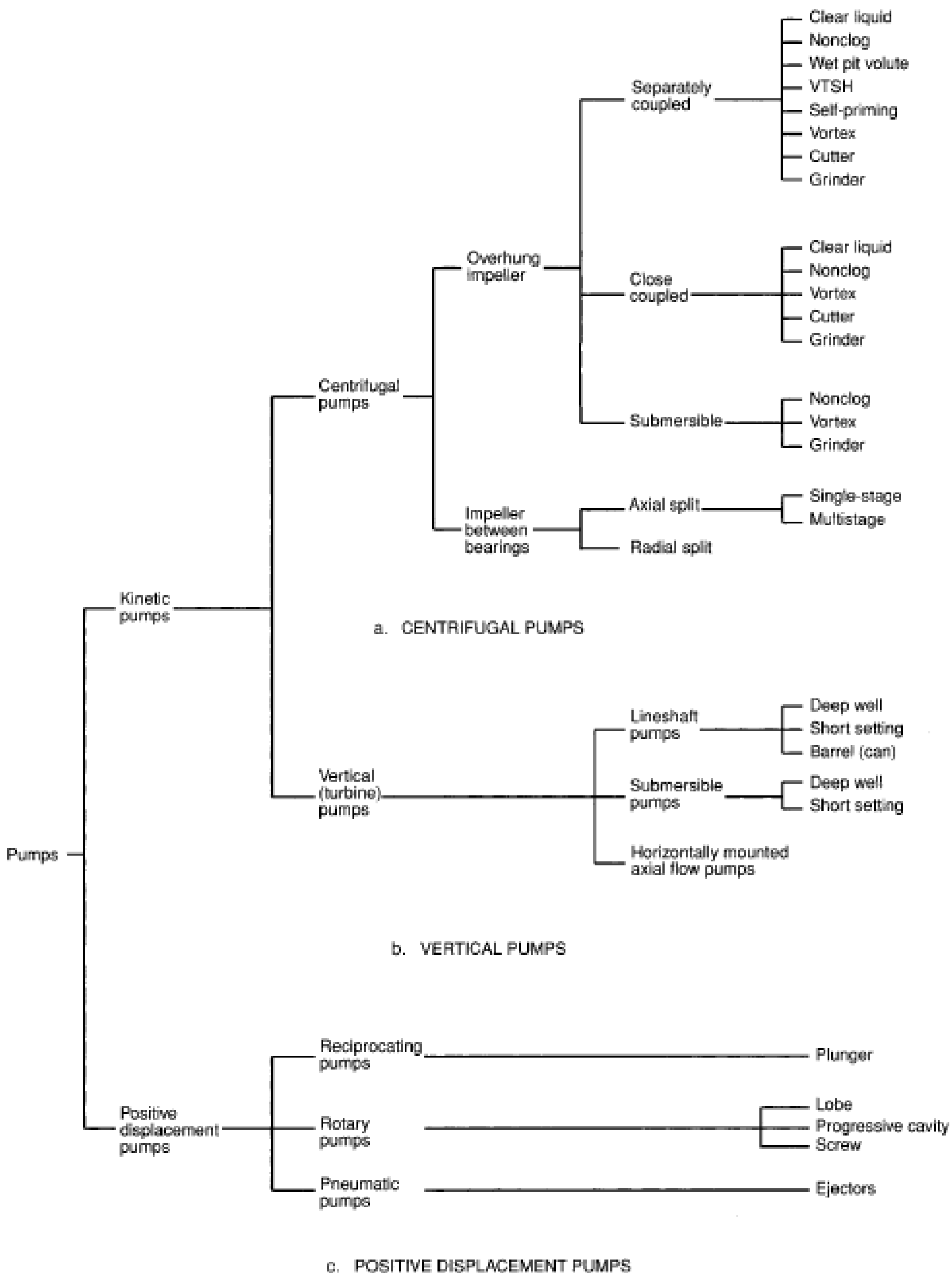


Figure E.1: Mechanical classification of pumps [10].

E.2 Flow rates through pipes

The flow rates through the different pipes are given in figure E.2:

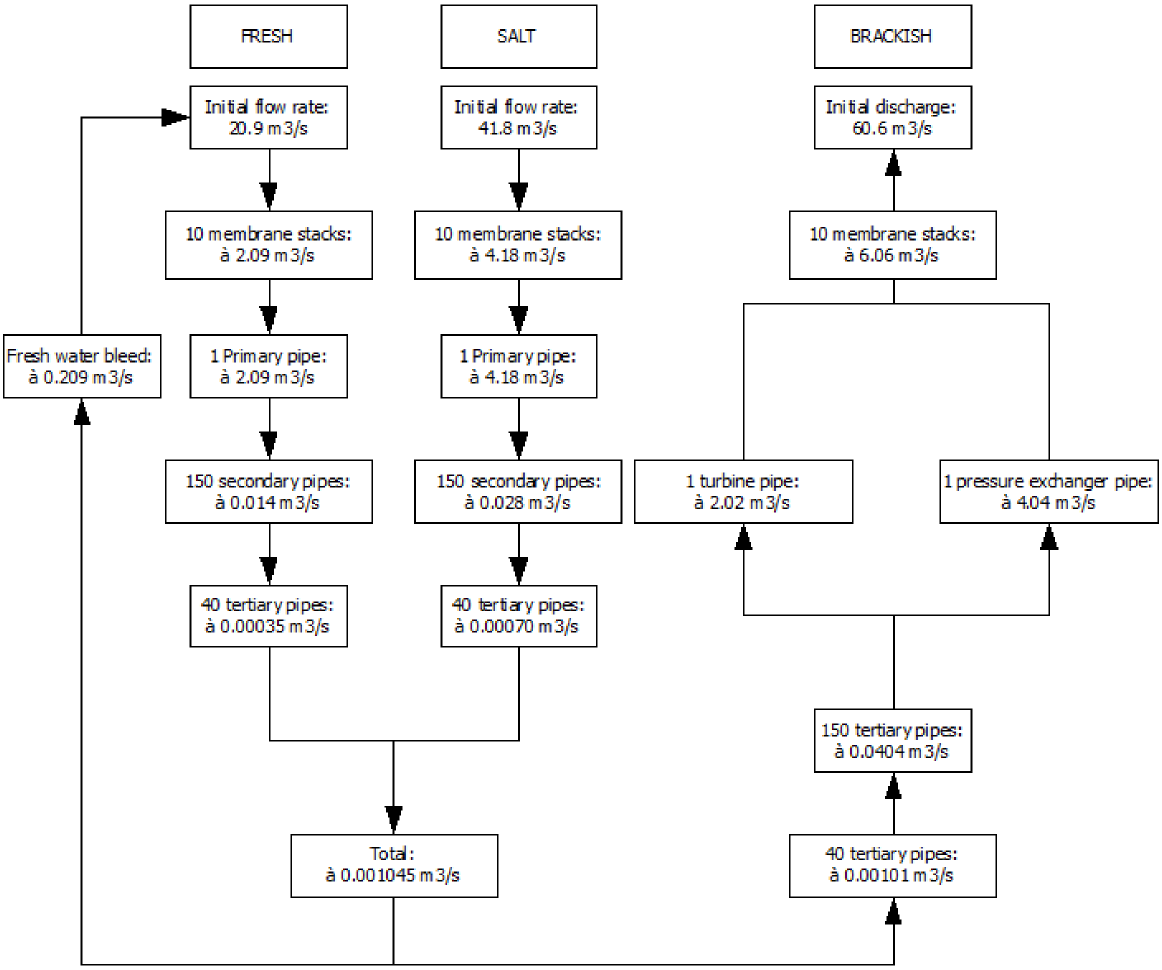


Figure E.2: Flow rates through the different pipes.

E.3 Calculations

The calculations conducted in this section are the calculations associated with the 25 MW PRO power plant. These calculations are similar to the calculations required for the other power plants.

E.3.1 Fresh water pipes

E.3.1.1 Primary fresh water pipe

The diameter of the fresh water pipe is determined by conducting an optimization. The diameter which results in a minimization of the summation of the capital costs and energy costs is the optimum diameter. The capital costs of pipes are determined by using a cost estimation formula of pipes [11]:

$$C_{pipes} = 500 \cdot D \cdot L \tag{E.1}$$

The energy costs are determined by using equation E.2:

$$C_{energy} = \frac{\Delta H \cdot Q \cdot \rho \cdot g}{\eta_{pump}} \cdot t_{hrs/yr} \cdot C_{€/kWh} \tag{E.2}$$

In which Δh is the total head loss which can be determined by using equation B.11.

In order to determine the energy costs the flow rate, length and the summation of the local losses should be determined first. The flow rates through the pipes are given figure E.2, the length and local losses are determined by using the assumed position given (figure 6.33) and the membrane stack configuration (figure 6.35 – 6.38). The loss factors are given in figure B.4.

The fresh water flow encounters a lot of branches in which the fresh water flow is divided in a branched water flow $Q_{branched}$ and a through water flow $Q_{through}$. A branch results in loss factors $\xi_{branced}$ and $\xi_{through}$. In the case study, it is assumed that the angle of a branch is always equal to 90° and that the diameter of the branch pipe is much smaller than the diameter of the through pipe. In figure E.3, the loss factors due to a branch are given:

aftakking onder	Q_a/Q Q_d/Q	0,0 1,0	0,1 0,9	0,2 0,8	0,3 0,7	0,4 0,6	0,5 0,5	0,6 0,4	0,7 0,3	0,8 0,2	0,9 0,1	1,0 0,0
$\delta = 90°$	ξ_a	0,98	0,92	0,87	0,84	0,84	0,87	0,93	1,00	1,08	1,18	1,27
$D_a = D$	ξ_d	0,03	-0,01	-0,03	-0,03	-0,02	0,02	0,07	0,14	0,21	0,28	0,35
$\delta = 90°$	ξ_a	1,3	1,4	1,5	1,8	2,4	3,2	4,3	5,6	7,2		
$D_a = 0{,}581 \cdot D$	ξ_d	0,15	-0,14	-0,15	-0,1	-0,05	0,00	0,05	0,13	0,2	0,25	0,3
$\delta = 90°$	ξ_a	1,0	1,6	3,0	5,3	8,9	13,8	19,5	25,3	31,3		
$D_a = 0{,}349 \cdot D$	ξ_d	0	0	0	0	0,1	0,1	0,1	0,1	0,2	0,2	0,2

Figure E.3: Loss factors due to a branch.

According to figure E.3, the loss due to the through flow is minimal for small branch pipe diameters. In the case study therefore, only a loss factor due to the branch flow is considered. The fresh water flow encounters the following changes in cross section or direction in its way to the membrane stacks. The loss factors of the fresh water flow to the top of the secondary fresh water pipe given figure 6.36 are:

- 2 x 90° bend: 2.60
- 1 x branch loss: 1.00
- Total loss: 3.60

The length of the primary fresh water pipe varies for each membrane stack and can be estimated by using figure 6.33. Now the flow rate, length and the total local loss are known, the optimum pipe diameter can be determined by minimizing the summation of the capital and energy costs (see figure 6.40). The results are given in table E.1:

	Pipe length	**Optimum diameter**	**Total head loss**	**Total head loss (D=1.150 mm)**
Stack #1	500 m	1.150 mm	2.81 m	2.81 m
Stack #2	460 m	1.150 mm	2.66 m	2.66 m
Stack #3	420 m	1.150 mm	2.51 m	2.51 m
Stack #4	380 m	1.150 mm	2.36 m	2.36 m
Stack #5	340 m	1.150 mm	2.21 m	2.21 m
Stack #6	300 m	1.200 mm	1.69 m	2.06 m
Stack #7	260 m	1.200 mm	1.57 m	1.92 m
Stack #8	220 m	1.200 mm	1.46 m	1.77 m
Stack #9	180 m	1.250 mm	1.11 m	1.62 m
Stack #10	140 m	1.250 mm	1.02 m	1.47 m

Table E.1: Optimum primary fresh water pipe diameter.

For simplicity, one diameter for the fresh water pipe is used. The used diameter is equal to 1.150 mm.

E.3.1.2 Secondary fresh water pipe

The diameter of the secondary fresh water pipe is determined by using the ratio between the flow rate through primary and secondary the pipe:

$$
\begin{aligned}
Q_{secondary} &= \frac{1}{n_{branches}} Q_{primary} \\
n_{branches} &= 150 \\
150 \cdot A_{secondary} &= A_{primary} \\
150 \cdot D^2_{secondary} &= D^2_{primary} \\
D_{secondary} &= \sqrt{\frac{1}{150} \cdot D^2_{primary}} = 0.094 = 100\,mm
\end{aligned}
\tag{E.3}
$$

The diameter of the secondary fresh water pipe is equal to 100 mm. The length of the secondary fresh water pipe is constant for each membrane stack and is equal to 15 m.

E.3.1.3 Tertiary fresh water pipe

The diameter of the tertiary fresh water pipe is determined by using the ratio between the flow rate through the secondary and tertiary pipe:

$$Q_{tertiary} = \frac{1}{n_{branches}} Q_{secondary}$$
$$n_{branches} = 40$$
$$40 \cdot A_{secondary} = A_{primary} \qquad \text{(E.4)}$$
$$40 \cdot D^2_{secondary} = D^2_{primary}$$
$$D_{secondary} = \sqrt{\frac{1}{40} \cdot D^2_{primary}} = 0.0158 = 16\,mm$$

The diameter of the tertiary fresh water pipe is equal to 16 mm. The length of the tertiary fresh water pipe is constant for each membrane stack and is equal to 0.25 m.

E.3.1.4 Capital costs

The capital costs of the primary fresh water pipe are given in table E.2

	Pipe length	Diameter	Capital costs
Stack #1	500 m	1.150 mm	€287.500,-
Stack #2	460 m	1.150 mm	€264.500,-
Stack #3	420 m	1.150 mm	€241.500,-
Stack #4	380 m	1.150 mm	€218.500,-
Stack #5	340 m	1.150 mm	€195.500,-
Stack #6	300 m	1.150 mm	€172.500,-
Stack #7	260 m	1.150 mm	€149.500,-
Stack #8	220 m	1.150 mm	€126.500,-
Stack #9	180 m	1.150 mm	€103.500,-
Stack #10	140 m	1.150 mm	€80.500,-

Table E.2: Capital costs primary fresh water pipe.

The capital costs of the secondary and tertiary fresh water pipes are:

$$C_{secondary} = 500 \cdot D \cdot L \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.1 \cdot 15 \cdot 150 \cdot 10 = €1.125.000,-$$
$$C_{tertiary} = 500 \cdot D \cdot L \cdot n_{tertiary} \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.016 \cdot 0.25 \cdot 40 \cdot 150 \cdot 10 = €120.000,- \qquad \text{(E.5)}$$

The total capital costs of the fresh water pipes is €3.085.000,-.

E.3.2 Salt water pipes

The diameter of the different salt water pipes is determined in the same manner as the fresh water pipes. The flow rate though the primary salt water pipe is given in figure E.2, the length is for each stack constant. The loss factors of the salt water flow from the pump to the pressure vessels are:

- 5 x 90° bend: 6.5
- 2 x branch: 2.0

- Total: 8.5

The results are given in table E.3:

	Pipe length	Optimum diameter	Total head loss
Primary	160 m	2.050 mm	0.89 m
Secondary	25 m	170 mm	
Tertiary	0.75 m	27 mm	

Table E.3: Optimum salt water pipe diameters.

The capital costs of the salt water pipes are:

$$\begin{aligned} C_{primary} &= 500 \cdot D \cdot L \cdot n_{stacks} = 500 \cdot 2.05 \cdot 160 \cdot 10 = €1.640.000,- \\ C_{secondary} &= 500 \cdot D \cdot L \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.17 \cdot 25 \cdot 150 \cdot 10 = €3.187.500,- \\ C_{tertiary} &= 500 \cdot D \cdot L \cdot n_{tertiary} \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.027 \cdot 0.75 \cdot 40 \cdot 150 \cdot 10 = €607.500,- \end{aligned} \quad \textbf{(E.6)}$$

The total capital costs of the salt water pipes is €5.435.000,-.

E.3.3 Brackish water pipes

The results of the brackish water pipes are given in table E.4:

	Pipe length	Optimum diameter
Primary pressure exchanger	225 m	1.700 mm
Primary turbine	95 m	1.200 mm
Secondary pressure exchanger	10 m	170 mm
Secondary turbine	16 m	170 mm
Tertiary	0.5 m	27 mm

Table E.4: Optimum brackish water pipe diameters.

The capital costs of the brackish water pipes are:

$$\begin{aligned} C_{primary;p.e.} &= 500 \cdot D \cdot L \cdot n_{stacks} = 500 \cdot 1.7 \cdot 225 \cdot 10 = €1.912.500,- \\ C_{primary;tur.} &= 500 \cdot D \cdot L \cdot n_{stacks} = 500 \cdot 1.2 \cdot 95 \cdot 10 = €570.000,- \\ C_{secondary;p.e.} &= 500 \cdot D \cdot L \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.17 \cdot 10 \cdot 100 \cdot 10 = €850.000,- \\ C_{secondary;tur.} &= 500 \cdot D \cdot L \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.17 \cdot 16 \cdot 50 \cdot 10 = €680.000,- \\ C_{tertiary} &= 500 \cdot D \cdot L \cdot n_{tertiary} \cdot n_{secondary} \cdot n_{stacks} = 500 \cdot 0.027 \cdot 0.5 \cdot 40 \cdot 150 \cdot 10 = €405.000,- \end{aligned} \quad \textbf{(E.7)}$$

The total capital costs of the brackish water pipes is €4.417.500,-.

E.3.4 Energy loss due to water transportation

The energy loss due to the water transportation is:

$$E_{pump} = \frac{\Delta H \cdot Q \cdot \rho \cdot g}{\eta_{pump}} \cdot t_{hrs/yr} \quad \textbf{(E.8)}$$

In equation E.8, the term Δh represents the required water elevation. The required elevation is the difference between:

1. The summation of optimum power plant level, stack height and head loss due to the water transportation.
2. The water level of the source minus the head loss due to the water intake (=1 m).

The water level of the Haringvliet is NAP + 0.5 m. With a head loss equal to 1 m due to the water intake, the water level in the fresh water pump sump is NAP – 0.5 m. The optimum power plant level (see section B.3) is NAP + 2.9 m, and the stack height is 6 m. The head loss due to the water transportation varies for each stack, so the energy loss of each stack varies as well. The energy loss for the fresh water flow is given in table E.5:

	Pipe length	Head loss	Water elevation	Energy loss
Stack #1	500 m	2.81 m	12.21 m	3.23 GWh/yr
Stack #2	460 m	2.66 m	12.06 m	3.19 GWh/yr
Stack #3	420 m	2.51 m	11.91 m	3.15 GWh/yr
Stack #4	380 m	2.36 m	11.76 m	3.11 GWh/yr
Stack #5	340 m	2.21 m	11.61 m	3.07 GWh/yr
Stack #6	300 m	2.06 m	11.46 m	3.03 GWh/yr
Stack #7	260 m	1.92 m	11.32 m	2.99 GWh/yr
Stack #8	220 m	1.77 m	11.17 m	2.95 GWh/yr
Stack #9	180 m	1.62 m	11.02 m	2.91 GWh/yr
Stack #10	140 m	1.47 m	10.87 m	2.87 GWh/yr

Table E.5: Energy loss fresh water flow.

The total energy loss due to the fresh water transportation is equal to 30.50 GWh/yr.

The water level of Lake Grevelingen is at NAP. With a head loss equal to 1 m due to the water intake, the water level in the salt water pump sump is NAP – 1.0 m. The head loss due to the water transportation is constant 0.89 m (see table E.3). The water elevation for each stack is therefore 10.79 m. The energy loss for the salt water flow is equal to 58.46 GWh/yr.

E.3.5 Turbine and generator efficiency loss

The turbine efficiency loss is equal to:

$$E_{loss;tur} = E_{initial} \cdot (1 - \eta_{tur}) = 30.6\,GWh/yr \quad \textbf{(E.9)}$$

E.4 200 MW PRO power plant

This section contains a brief calculation of the energy loss and capital costs in case of a 200 MW PRO power plant. The power plant configuration is given in figure E.4:

Figure E.4: Power plant configuration 200 MW PRO power plant.

The configuration of the 200 MW PRO power plant consists of 20 lines. Each line contains 4 rows membrane stacks. The diameters of the different pipes are the same as used in the case of the 25 MW PRO power plant. The length of the primary pipes differs, but the lengths of the secondary and tertiary pipes are equal to the lengths used in the case of the 25 MW PRO power plant. This results in a fixed costs per line (4 membrane stacks) of the secondary and tertiary pipes equal to €2.100.000,-. The length of the primary pipes is:

- Primary fresh row 1 and 4: 460 m
- Primary fresh row 2 and 3: 185 m
- Primary salt row 1 and 4: 185 m
- Primary salt row 2 and 3: 460 m

The length of the primary brackish pipe (pressure exchanger and turbine) varies for each line. The relation between membrane stack line and length is:

$$L_{primary\,brackish} = 100 + \left(n_{line} - 1\right) \cdot 60 \qquad \textbf{(E.10)}$$

The length of the primary pipes, together with the same total loss factor as used in the case of the 25 MW PRO power plant, results in the total head loss:

- Fresh head loss row 1 and 4: 2.66 m
- Fresh head loss row 2 and 3: 1.64 m
- Salt head loss row 1 and 4: 1.47 m
- Salt head loss row 2 and 3: 1.20 m

The head losses result in the required water elevation:

- Fresh water elevation row 1 and 4: 12.06 m
- Fresh water elevation row 2 and 3: 11.04 m
- Salt water elevation row 1 and 4: 11.37 m
- Salt water elevation row 2 and 3: 11.10 m

Now all the parameters are known, the total capital costs and energy loss in case of the 200 MW PRO power plant can be determined. The relation between total capital costs and power plant capacity is given in figure E.5. Each line of 4 membrane stacks represents a capacity of 10 MW.

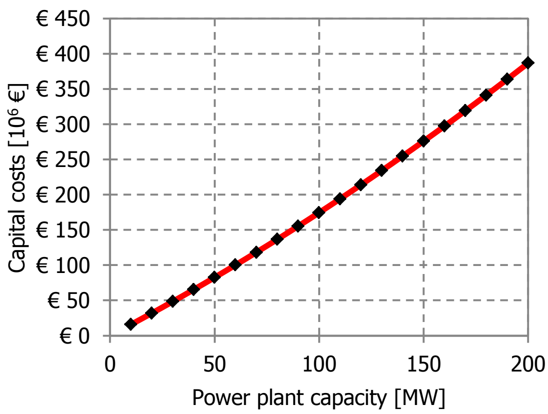


Figure E.5: Relation between power plant capacity and capital costs of the pumps, pipes and turbines.

The total capital costs in case of the 200 MW PRO power plant are about €385.000.000,-.

The relation between the total energy loss and power plant capacity is given in figure E.6:

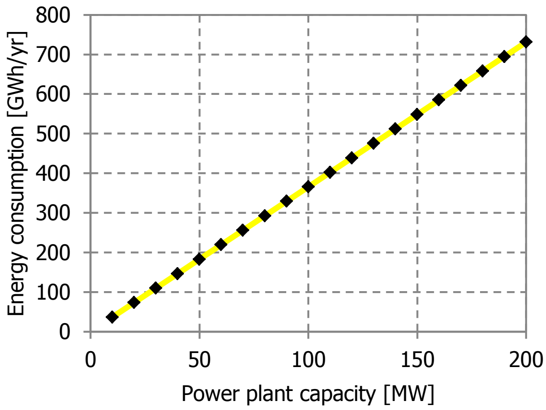


Figure E.6: Relation between power plant capacity and energy loss due to water transportation.

The energy loss due to the water transportation in case of the 200 MW PRO power plant is about 730 GWh/yr.

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